

CONTRACT CHANGE ORDER

Change Requested by: Engineer

CCO: 176 Suppl. No. 0 Contract No. 04 - 0120F4 Road SF-80-13.2/13.9 FED. AID LOC.:

To: **AMERICAN BRIDGE/FLUOR ENTERPRISES INC A JOINT VENTURE**

You are directed to make the following changes from the plans and specifications or do the following described work not included in the plans and specifications for this contract.

NOTE: This change order is not effective until approved by the Engineer.

Description of work to be done, estimate of quantities and prices to be paid. (Segregate between additional work at contract price, agreed price and force account.) Unless otherwise stated, rates for rental of equipment cover only such time as equipment is actually used and no allowance will be made for idle time. This last percentage shown is the net accumulated increase or decrease from the original quantity in the Engineer's Estimate.

Adjustment of Compensation at Lump Sum:

This CCO accepts as "fit for purpose" the localized misalignments and welding transitions found at the transverse welded field splice joints of the Orthotropic Box Girders (OBG) steel deck skin plates as depicted in the attached technical report - "Behavior of Complete Joint Penetration Field Welds Between Orthotropic Box Girder Segments," dated October 28, 2010. The attached report documents the conditions found in the field, and serves as the accepted technical document addressing the alignment issues and the necessary work to mitigate the local misalignment of the field joints in lifts 1 through 14 of the OBG.

Adjustment of Compensation at Lump Sum.....\$0.00

Estimated Cost: Increase ☐ Decrease ☐ \$0.00

By reason of this order the time of completion will be adjusted as follows: 0 Days

Submitted by

Signature

Resident Engineer

Kannu Balan, Senior T.E.

Date 2-1-11

Approval Recommended by

Signature

Supervising Bridge Engineer

William Casey, Sup. B.E.

Date 2-1-11

Engineer Approval by

Signature

Principal Transportation Engineer

Peter Siegenthaler, Prin. T.E.

Date 2-8-11

We the undersigned contractor, have given careful consideration to the change proposed and agree, if this proposal is approved, that we will provide all equipment, furnish the materials, except as otherwise be noted above, and perform all services necessary for the work above specified, and will accept as full payment therefor the prices shown above.

NOTE: If you, the contractor, do not sign acceptance of this order, your attention is directed to the requirements of the specifications as to proceeding with the ordered work and filing a written protest within the time therein specified.

Contractor Acceptance by

Signature

(Print name and title)

Date

BRIAN A. PETERSEN - PROJECT DIRECTOR 04 FEB 11

DL McQUAID & ASSOCIATES Inc.

2306 Cassidy Drive
Bethel Park, PA 15102
Phone No. 412-831-9177
Fax No. 412-851-4116

To: American Bridge / Fluor Enterprises Inc., JV
Mr. Mike Flowers
Project Director
375 Burma Road
Oakland, CA 94607

October 22, 2010

Phone No. 510-808-4600
Fax No. 510-808-4601

Re: / Review of Expert Report by John M. Barsom and Kevin M. Smith – Behavior of Complete Joint Penetration Field Welds Between Orthotropic Box Girder Segments

Dear Mr. Flowers:

I have reviewed the Technical Report – Behavior of Complete Joint Penetration Field Welds Between Orthotropic Box Girder Segments by John M. Barsom and Kevin M. Smith as requested and I offer the following letter in support of the technical content and accuracy of the report. Based on this review I recommend that the report be used by all parties to accept the misalignment of the orthotropic box girders.

I provided Mr. Barsom information as he required in the development of the expert report and have discussed with him the techniques used to fabricate, erect and weld the orthotropic box girders at the ZPMC facility in China and at the project site in Oakland California. I also visited the project site and witnessed the fit-up and welding and can verify that the field welding was done as described in the expert report and is in general conformance with the requirements of the AWS D1.5 Bridge Welding Code and the workmanship standards expected of a modern day steel erector. I have reviewed the stress calculations and they appear to be acceptable, however, that is not my area of expertise and I will not comment on them in detail. The expert report has addressed the alignment issues in depth and has provided the technical information to justify accepting the orthotropic box girders for the SAS Bridge.

Based on my discussions with American Bridge/Fluor, Mr. Barsom and my involvement as a welding consultant to the Joint Venture it is my professional opinion based on a reasonable degree of engineering certainty that the expert report accurately describes the condition of the fit-up and welding at the project site, is technically accurate and I recommend that it be accepted as written and submitted to Caltrans for their approval.

Sincerely,

D. L. McQuaid P.E.

D. L. McQuaid P.E.
Welding Consultant

**BEHAVIOR OF COMPLETE JOINT PENETRATION FIELD WELDS BETWEEN
ORTHOTROPIC BOX GIRDER SEGMENTS**


SAN FRANCISCO-OAKLAND BAY BRIDGE

SELF-ANCHORED SUSPENSION SPAN

BY

John M. Barsom

Barsom Consulting Ltd


11/11/2010

And

Kevin M. Smith

American Bridge / Fluor Enterprises Inc., A Joint Venture


11/12/10

INTRODUCTION

Because it is a "self-anchored" suspension bridge most of the orthotropic box girder deck structure will be under significant axial compression when completed. This compression eliminates, or minimizes, concern regarding the fatigue performance of the transverse field deck splices that are under compression. This is evidenced by the Contract Document provision that permits transverse backing bars used for the complete joint penetration (CJP) field welds to remain in place for all but the two most eastern deck splices – those between Lifts 12 and 13 and those between Lifts 13 and 14. Bridges with orthotropic deck plates commonly have the transverse backing removed to minimize potential fatigue damage. Compression in the eastern portion of the SFOBB structure is reduced as a result of shear lag and dead and live load flexure stresses.

The American Welding Society (AWS) D1.5-2002 Bridge Welding Code is the governing document for the welded field splices. It states in Section 3.3.3: "Where parts are effectively restrained against bending due to eccentricity in alignment, the offset from theoretical alignment shall not exceed 10 percent of the thickness of the thinner part joined, but in no case shall be more than 3mm [1/8 in.]" Furthermore, it states in Section 3.3.1.1: "The separation between faying surfaces of plug and slot welds, and butt joints landing on a backing, shall not exceed 2 mm [1/16 in.]"

The following discusses the fatigue implications of deck plate misalignments at the field splices greater than those allowed by AWS D1.5, and addresses backing bar separations that are more than 2mm.

DISCUSSION

Dimensional tolerances specified for the fabrication of the steel box girders allow variations in the overall depth, skin plate flatness, and the placement and alignment of stiffeners and other components between adjacent segments. As an example, the specified tolerance for the depth of box girder segments is +8mm/-5mm. In order to achieve a “best fit” between segments significant effort has been expended during both trial and in-field fit-up to accommodate the dimensional variations. For an overview of the welding and bolting operation used to align and connect the Orthotropic Box Girder top deck plates in the field, reference Appendix B. Notwithstanding the efforts to align the field splices, offsets greater than those allowed by AWS D1.5 have occurred over localized areas of the deck structure.

For the transverse deck plate field splices completed to date for OBG Lifts 1E through 7E and 1W through 6W, the plates are typically aligned within AWS D1.5 tolerances of 2mm; however, at a few locations the offsets exceed the 2mm alignment tolerance by up to 4mm. Reference the offset maps provide in Appendix C. The average offset between adjacent deck plates is 0.33mm with a standard deviation of 1.15mm. The accumulated out-of-tolerance deck misalignments along a box girder splice are, on average, not more than 7% of the deck width, with a peak offset of 6mm. The average length of the out-of-tolerance sections is less than 400mm. Because the deck plate is highly restrained at the north and south edges of the box girder where the deck forms a corner with the adjoining edge plate, there is a higher frequency of out-of-tolerance deck misalignments at these locations. These misalignments are located out of the traveled way where fatigue from direct wheel loads is not a concern and where the thickness of the deck plate is 20mm, contrary to the majority of the deck comprising the roadway shoulders and traveled way which is 14mm thick, except at the far west and east ends where it thickens to 20mm.

In some instances the planar misalignment has prevented a tight fit between the transverse backing bar and the underside of the deck plates causing maximum separations of up to 4mm over short lengths. A procedure has been developed to fill these gaps using a shielded metal arc process prior to implementing flux-cored and submerged arc welding for the completion of the CJP groove welds.

When field welding at a transverse deck splice is complete the weld reinforcement is ground flush. In areas of misalignment grinding has produced sloped transition surfaces that are no steeper than 2.5 to 1. The AASHTO fatigue category for field splices that do not have the backing removed, with or without sloped transitions, and regardless if the crown is ground flush or not, is category “C”¹. This is the fatigue category representing all but the two eastern field splices. If the weld reinforcement, or backing, is removed and the welds are ground to a slope not exceeding 2.5 to 1 the fatigue category is “B”¹. This is the category that will apply to the two most eastern deck splices and will provide the highest fatigue category for a welded member – category “A” being reserved for plain members comprised of base metal with rolled, thermal cut, or cleaned surfaces.

Planar misalignments at the deck splices may result in secondary stresses due to the resultant vertical eccentricity between the adjoining plates. As noted above, the planar misalignments are localized, occurring over short lengths and average no more than 7% of the deck width. The unfactored axial compression stresses from dead load and live load, combined with the secondary

flexural stresses due to a planar misalignment of 6mm have been calculated (reference Appendix D) to be 385MPa in the 14mm thick deck plate. While this stress is larger than the minimum required yield strength of the material, 345MPa, it should be noted that the actual yield strength of the 14mm plate varies between 364MPa and well above 400MPa with an average yield strength of 432MPa. Because of the higher yield strength yielding of the 14mm thick deck plate is not expected. However, because the misalignments are over short lengths and when combined represent only a small percentage of the overall deck width, if yielding were to occur at discrete locations, the resulting strain would result in a redistribution of stress that would mitigate any adverse impacts. Additionally, the reduction in stress due to the presence of the backing bar, the 2.5 to 1 sloped transitions and the 50mm epoxy asphalt concrete overlay have not been considered here but are suspected to reduce the stress in the misaligned portions of the deck plate. Therefore, secondary stresses due to a planar misalignment of 6mm have been determined to be within acceptable limits for service loads.

Fatigue Behavior at the Weld Root with Backing – Deck in Compression

Typically, solidification of deposited weld metal induces residual compressive stresses at the root of single-V CJP welds, such as those for the deck splices. Fatigue cracks rarely initiate under compression-compression cyclic loading. The size of these cracks, should they occur, are limited to the size of the plastic zone at the location of any stress riser. The crack size will be limited to a few thousands of an inch, a value less than what the code allows for discontinuities in base and weld metals that are inspected by ultrasonic testing. Furthermore, because the transverse splices are under significant axial compression, if cracks were to initiate they would not propagate under cyclic compressive stresses. This case applies to all but the two most eastern deck field splices.

Fatigue cracks, if they were to initiate between the deck plate and the backing bar, would initiate at the corner between the bottom surface of the deck plate and the root of the deposited weld metal. The attached figure in Appendix A presents a typical cross section of deck plates, backing bar, shielded metal arc welds with a nominal gap between the bottom surface of the deck plate and the top surface of the backing bar. The dimensions of the 9/16-inch (14mm) thick deck plate, the 1½-inch wide backing bar and the cross section of the welds are proportioned to correctly represent the dimensions of the field welded joint. The figures indicate that a deck plate planar misalignment may produce a gap between the bottom surface of the deck plate and the top surface of the backing bar of up to 1/8-inch. The geometry at the corner between the bottom surface of the deck plate and the deposited weld bead does not change significantly when the gap varies from zero through 1/8-inch. Consequently, the stress concentration at this location is negligibly affected by the gap between steel backing and the deck plate. Therefore, the performance of the groove weld is not expected to be different for gaps ranging from 0mm to 4mm.

AASHTO LRFD Section 6.6.1.2.1² states that residual stresses shall not be considered in investigating fatigue and that the provisions for fatigue shall be applied only to details subjected to a net applied tensile stress. In regions where the unfactored permanent loads produce compression, fatigue shall be considered only if the compressive stress is less than twice the maximum tensile live load stress resulting from the fatigue load combination specified in Table 3.4.1-1. As demonstrated in the calculations provided in Appendix D, the dead load compressive

stresses for splices 1 through 10 range from 83MPa to 104MPa. These dead load compressive stresses are greater than twice the tensile live load stress of 40.5MPa. For splice 11 the dead load compressive stress is 72MPa. It is expected that consideration of the wheel load distribution due to the epoxy asphalt concrete overlay will reduce the tensile live load stress from 40.5MPa to less than 34MPa. Under these conditions, load-induced fatigue need not be considered for the deck welds in compression-compression cyclic loading.

Fatigue Behavior at the Crown of the Weld – Deck in Compression

Based on geometry alone – neglecting the fact that the deck plate is not subjected to cyclical tension stresses under normal operating conditions – in the absence of rejectable weld metal indications, the fatigue performance of the field splices will be dictated by the transition between the two plates. Fatigue cracks rarely initiate under compression-compression cyclic loading, however, if they were to form, they would initiate at the toe of the weld. Fatigue cracks propagate perpendicular to the direction of the applied cyclic stresses. Because of the inclined side of the single-V groove, fatigue cracks, if they were to occur, would propagate from the toe of the weld into the base metal of the top deck plate. However, the propagation would not proceed because the deck plate is not subjected to tension cyclic stresses under normal operating conditions.

The preceding discussion demonstrates that fatigue cracks should not initiate or propagate from the face of the groove welds between the top deck plates.

CJP groove welds with backing and excess weld reinforcement correspond to AASHTO fatigue category “C”. Though not accounted for, grinding the face of the weld to a 2.5 to 1 slope improves the geometry and fatigue resistance at the face of the weld. It should be emphasized that for all but the last two field splices these welds are subjected only to compressive cyclic stresses under normal operating conditions, a condition under which fatigue cracks rarely initiate and do not propagate.

Fatigue Behavior of Welds – Deck without Significant Compression

The box girder field splices at PP117.5 and PP124.5 are subjected to a significant amount of negative dead load moment which reduces the dead load compressive stresses in the deck plate. The deck plate compressive stresses are further reduced at these splices because the box girder cross sections are more robust than the typical box girder cross section. The area and moments of inertia are approximately 40% to 70% more at these cross sections than the typical bridge cross sections. Additionally, the deck plate thickness at these splices is 20mm instead of the 14mm thick deck plate used elsewhere.

At these field splices the backing bar will be removed and the 2.5 to 1 sloped transitions will be present at both the top and bottom surfaces of the deck plate, effectively eliminating any stress risers. The un-factored axial compression stresses from dead load and live load, combined with the secondary flexural stresses due to a planar misalignment of 6mm have been calculated to be 96MPa and 111MPa for the box girder field splices at PP117.5 and PP124.5, respectively. Therefore, secondary stresses at these splices have been determined not to be significant.

The stress range at these splices will be less than that previously calculated for the typical splices due to the reduced span length between floor beams and the increased deck plate thickness. The fatigue category for these splices is "B", which permits a higher Constant-Amplitude fatigue threshold.³ The fatigue stress range at PP117.5 and PP124.5 is 23MPa which is less than the nominal fatigue resistance of 55MPa. Therefore, load induced fatigue including secondary effects is acceptable for the two most eastern deck field splices.

Additional locations to be considered:

The Department has independently measured the planar alignment and has reported a few notable differences from the values provided in Appendix C. A maximum offset of 13mm has been identified at the intersection of the edge plate and the deck plate. This misalignment occurs at a single location over a length not more than 100mm. This offset is located outside the traveled way at the edge of the box girder where traffic live load is not present and fatigue is not a concern. The deck plate at this location is 20mm thick and is highly restrained due to the proximity of the misalignment to the edge plate. At this location the misaligned portion of the deck plate has been transitioned at 2.5:1 slopes on both the top and bottom of the deck plate and live loads are not present; therefore, stresses, including secondary stresses, at this location are not a concern.

The Department also noted a 7mm offset over an 800mm length at the field splice between Lift 1E and 2E and an 8mm offset over a 200mm length at the field splice between Lift 2E and 3E. These offsets are located within the traveled way and are more than the 6mm offset measured by Smith Emery Company, ABFJV's independent welding quality control inspector. At these locations, the un-factored axial compression stresses from dead load and live load, combined with the secondary flexural stresses due to a planar misalignment of 8mm have been calculated to be 259MPa and 387MPa in the 14mm thick deck plate (reference Appendix D). These stresses are equivalent to the maximums reported elsewhere on the bridge and have been determined to be within acceptable limits for service loads. Fatigue loading at these locations only increases slightly and the dead load compressive stresses are still greater than twice the tensile live load stress of 43.1MPa. Therefore, load-induced fatigue need not be considered for these deck welds in compression-compression cyclic loading.

CONCLUSION

The preceding discussion indicates that:

1. Compression stress in the deck plate and the compressive residual stress at the root of the single-V CJP weld will inhibit fatigue crack initiation and propagation at any gap between the backing bar and the deck plate.
2. Compression stress in the deck plate will prevent fatigue cracks at the face of the sound single-V CJP weld in areas of planar misalignment. If fatigue cracks were to initiate they would not propagate due to the compressive stress.
3. Groove-welded deck field splices between Lifts 12 and 13 and Lifts 13 and 14, where the backing is removed, the top and bottom faces ground, and transitions have slopes no steeper than 2.5 to 1, will have no stress risers. Secondary stresses have been determined not to be significant. Under these conditions the field splices will provide acceptable fatigue resistance.
4. The performance of the groove-welded deck field splices with backing bar gaps ranging from zero to 2mm, with additional 2mm tolerance, will not differ substantially from one another.

References:

1. AASHTO LRFD Bridge Design Specifications, Second Edition, 1998, Table 6.6.1.2.3-1 Detail Categories for Load-Induced Fatigue.
2. AASHTO LRFD Bridge Design Specifications, Second Edition, 1998, Section 6.6.1.2.1 Load-Induced Fatigue, Application.
3. AASHTO LRFD Bridge Design Specifications, Second Edition, 1998, Table 6.6.1.2.5-3 Constant-Amplitude Fatigue Thresholds.

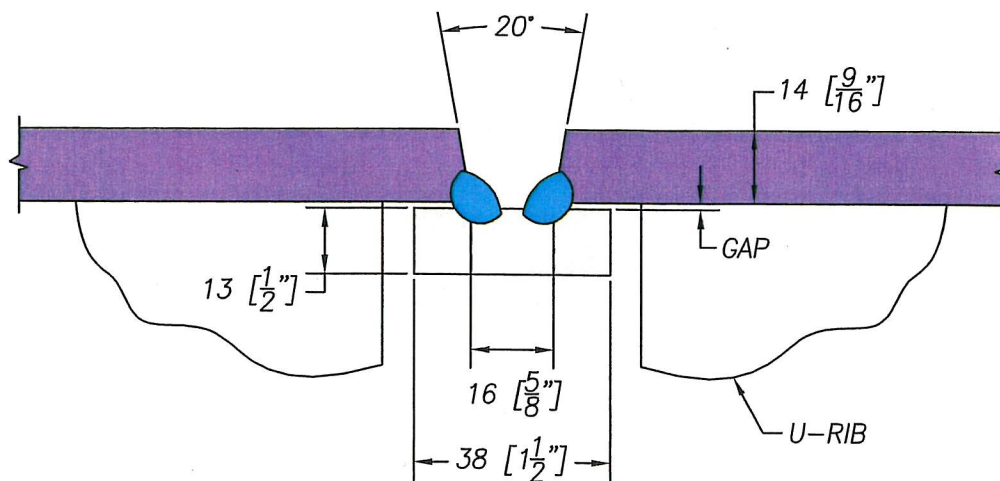
APPENDIX A

FIGURE 1
TYPICAL DECK JOINT CROSS SECTION (DECK PLATES ALIGNED)

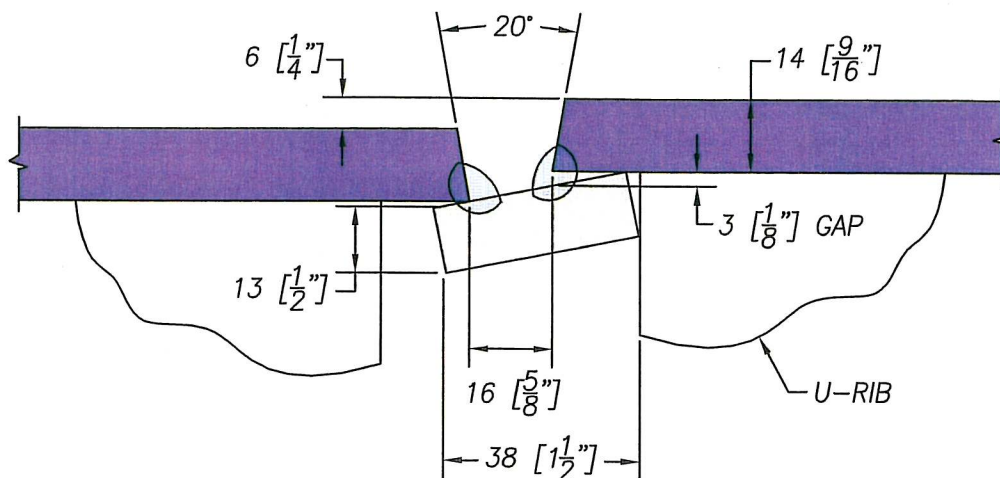


FIGURE 2
TYPICAL DECK JOINT CROSS SECTION (DECK ALIGNED TO 6mm)

SAN FRANCISCO OAKLAND BAY BRIDGE
 EAST SPAN SEISMIC SAFETY PROJECT
 SELF ANCHORED SUSPENSION BRIDGE
 (SUPERSTRUCTURE AND TOWER)

STATE OF CALIFORNIA
 DEPARTMENT OF TRANSPORTATION
 CONTRACT NO. 04-0120F4
 BRIDGE NO. 34-0006L/R
 DISTRICT 04 COUNTY SF ROUTE 80 KILOMETER POST 13.2 / 13.9



Design By: K. SMITH Date: 09/30/10
 Design Chk: D. HESTER Date: 09/30/10
 Drawn By: K. SMITH Date: 09/30/10
 Drawing Chk: D. HESTER Date: 09/30/10
 In Charge Of: J. CALLAGHAN

AB Job No. 660110

ORTHOTROPIC BOX GIRDER
 DECK JOINT CROSS SECTION

Scale:
 NTS

Sheet No.
 1 of 1

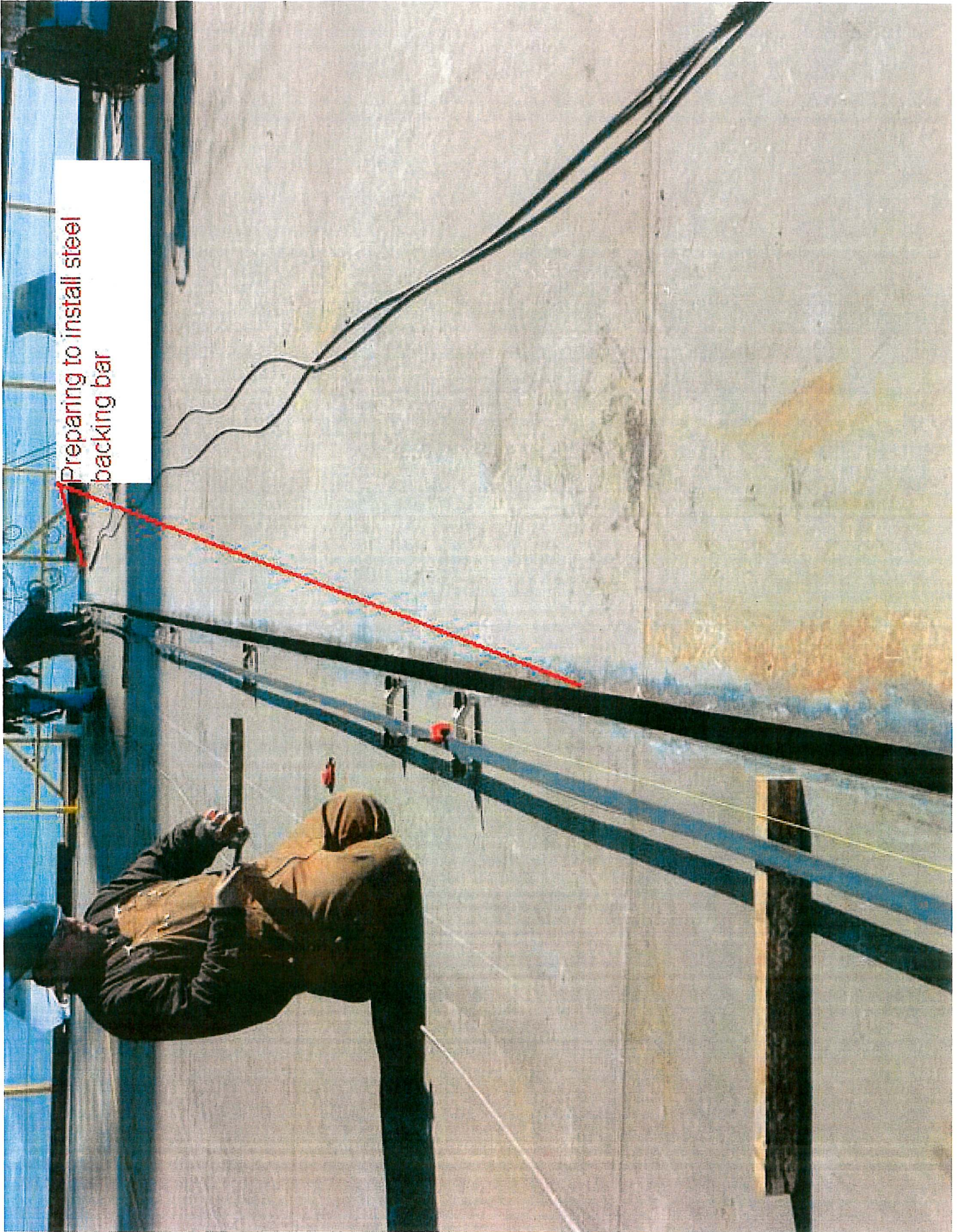
APPENDIX B

OVERVIEW OF THE BOX GIRDER BOLTING AND WELDING OPERATION

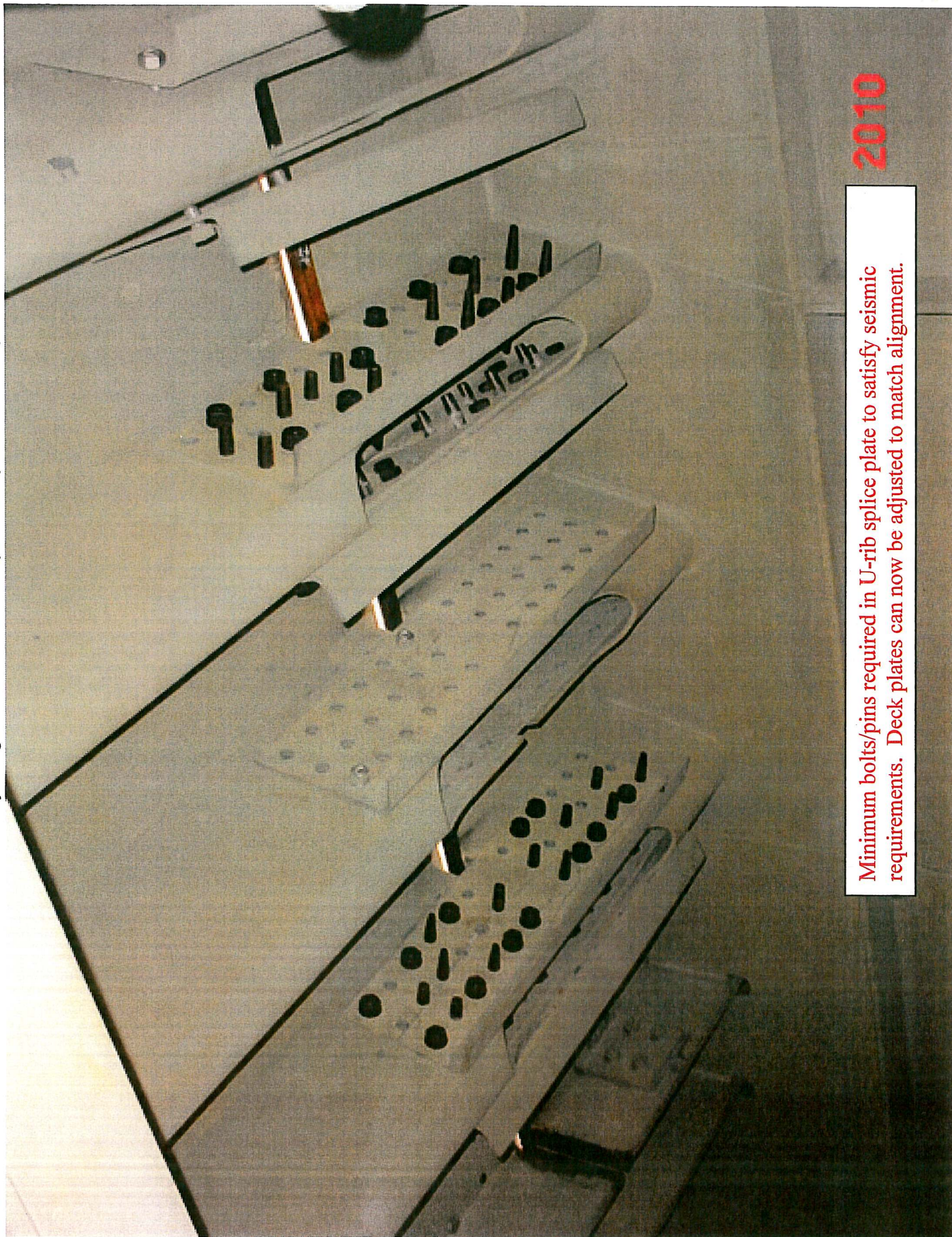
San Francisco-Oakland Bay Bridge: Self Anchored Suspension Span / Orthotropic Box Girder

Overview of welding and bolting operation for Top Deck Plates for Lifts 1-6

San Francisco-Oakland Bay Bridge: Self Anchored Suspension Span / Orthotropic Box Girder



San Francisco-Oakland Bay Bridge: Self Anchored Suspension Span / Orthotropic Box Girder



2010

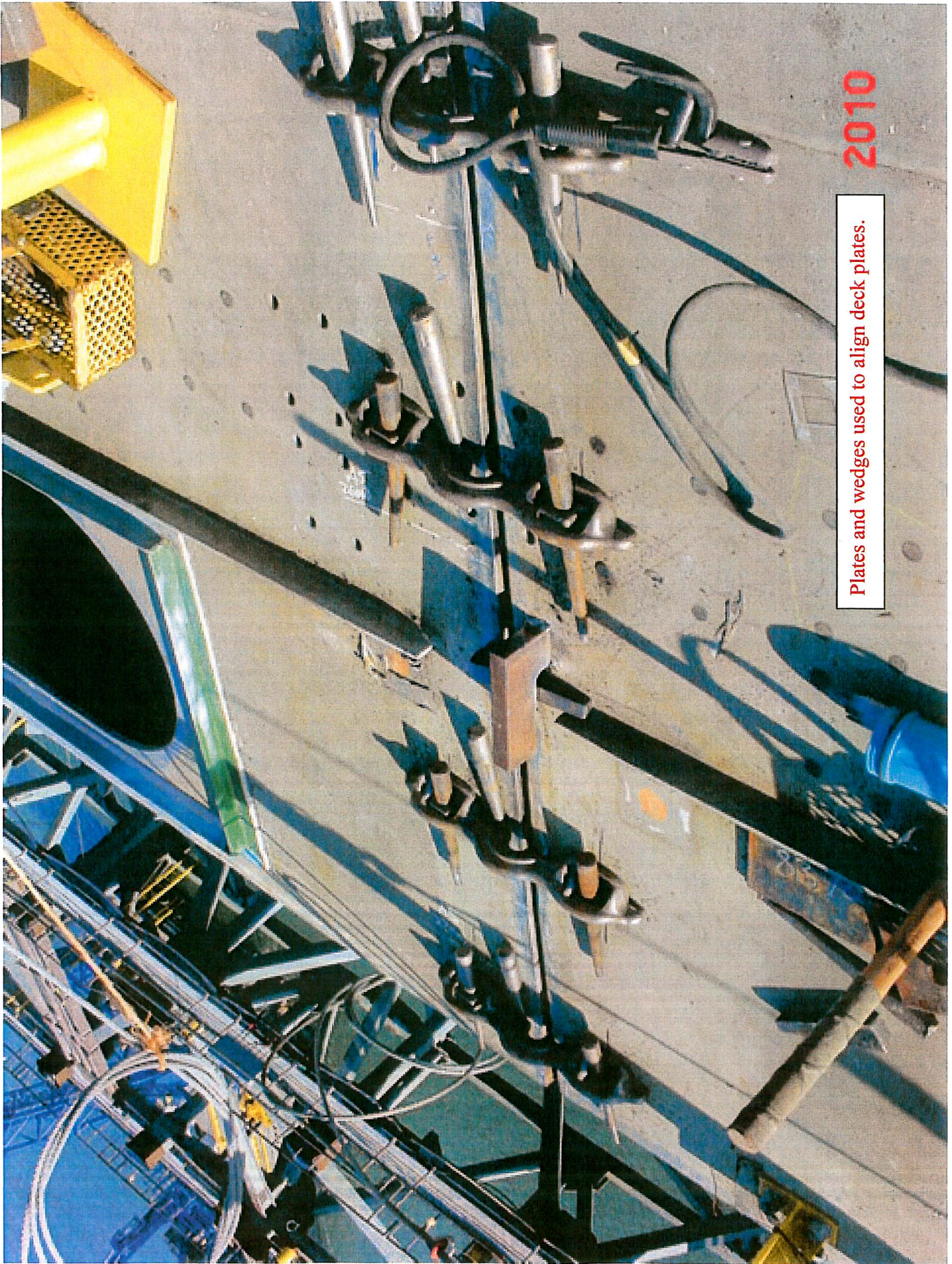
Minimum bolts/pins required in U-rib splice plate to satisfy seismic requirements. Deck plates can now be adjusted to match alignment.



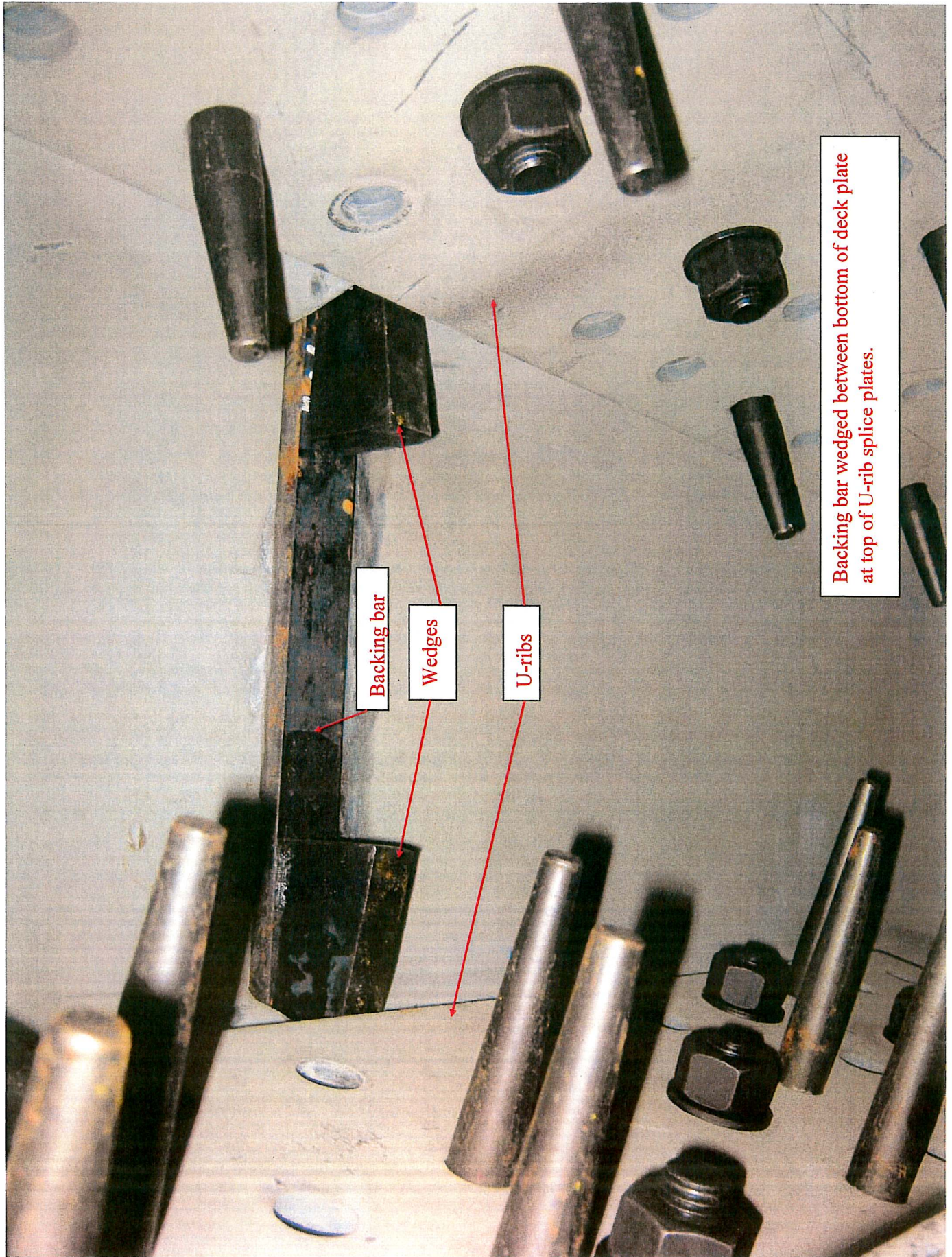
Bridge Cam / Welding gauge used to measure planer off-set.



San Francisco-Oakland Bay Bridge: Self Anchored Suspension Span / Orthotropic Box Girder



Plates and wedges used to align deck plates.





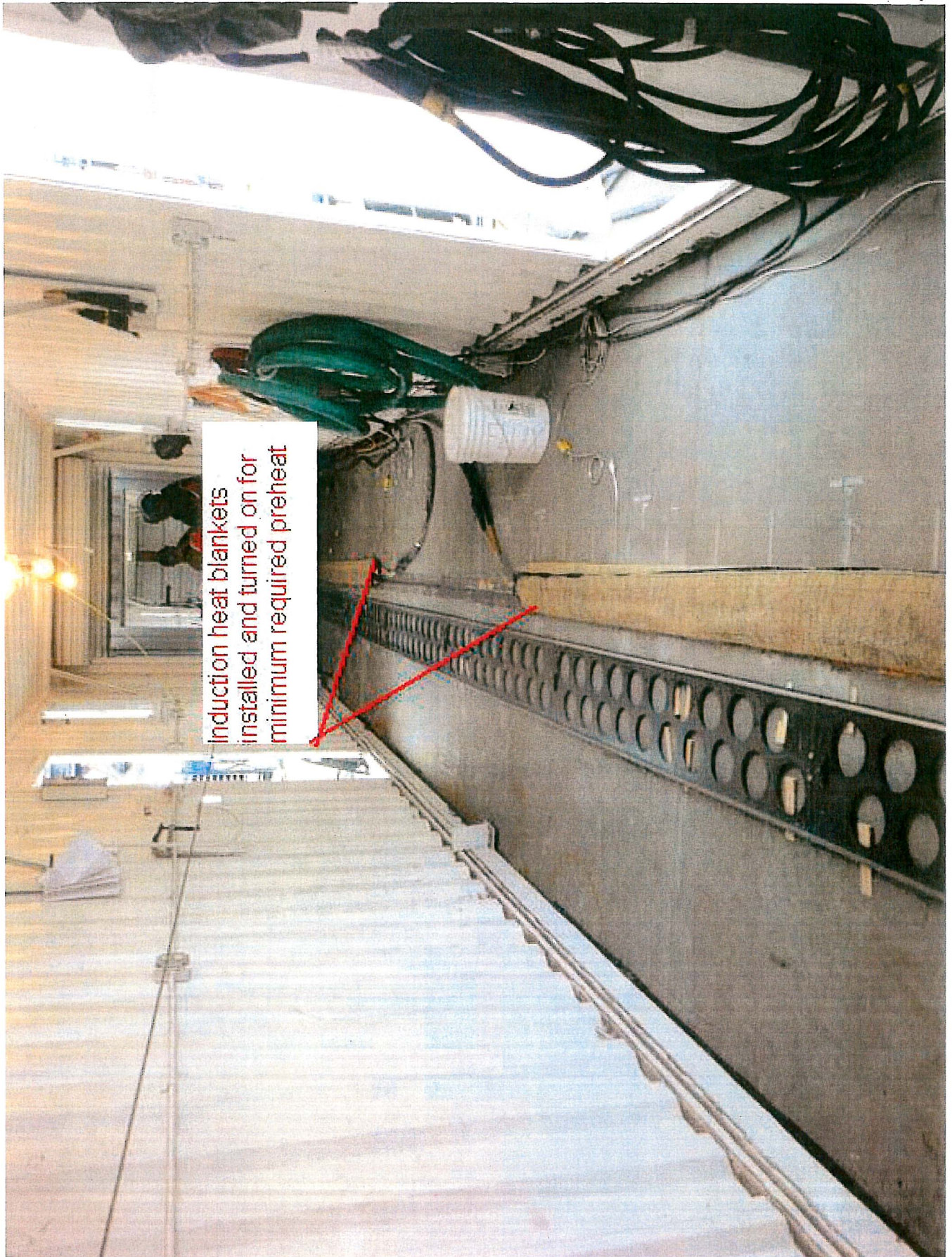
Typical amount of blots/pins installed in U-rib splice plates after deck plates are aligned and before welding begins.



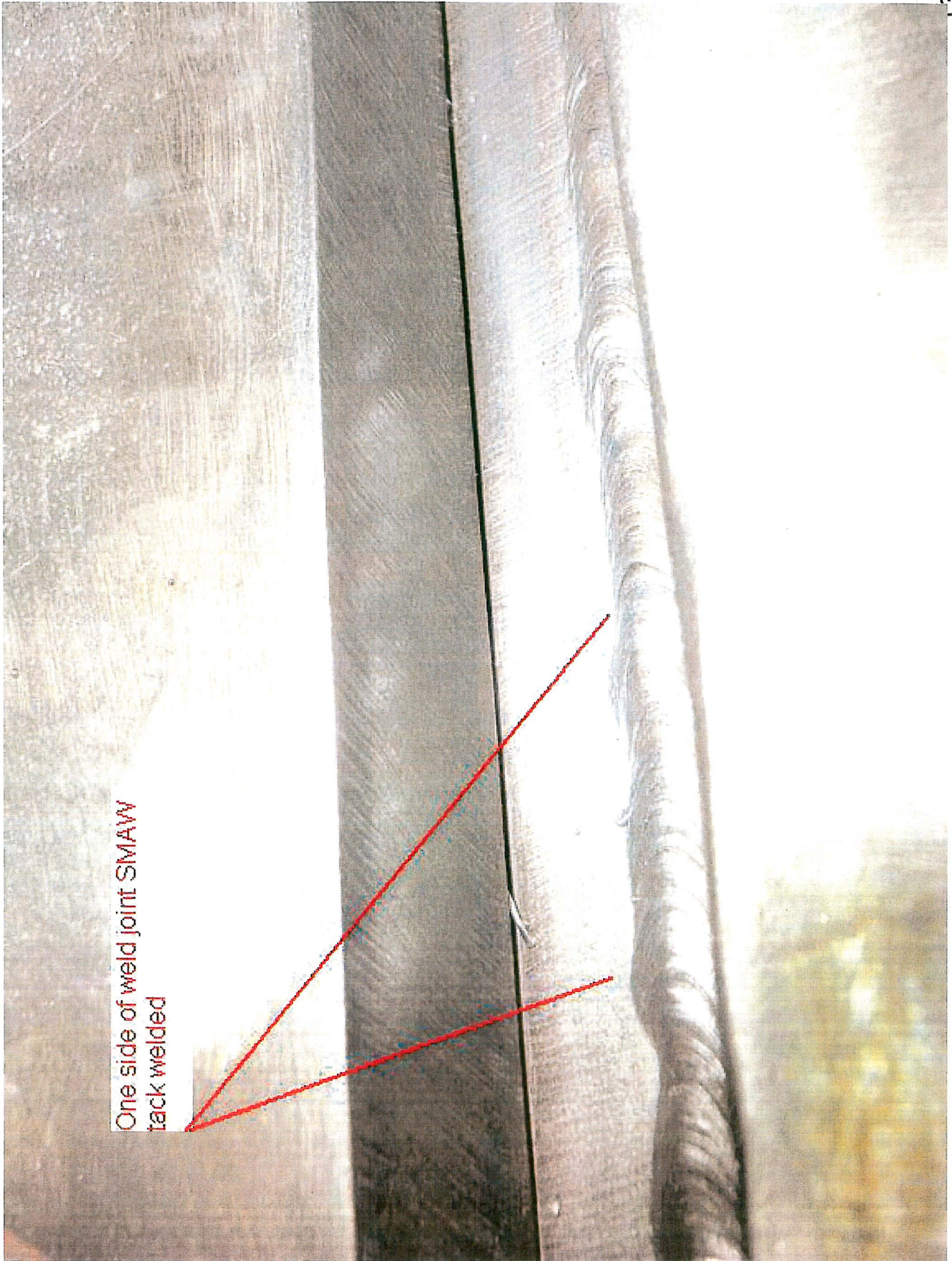
San Francisco-Oakland Bay Bridge: Self Anchored Suspension Span / Orthotropic Box Girder



San Francisco-Oakland Bay Bridge: Self Anchored Suspension Span / Orthotropic Box Girder

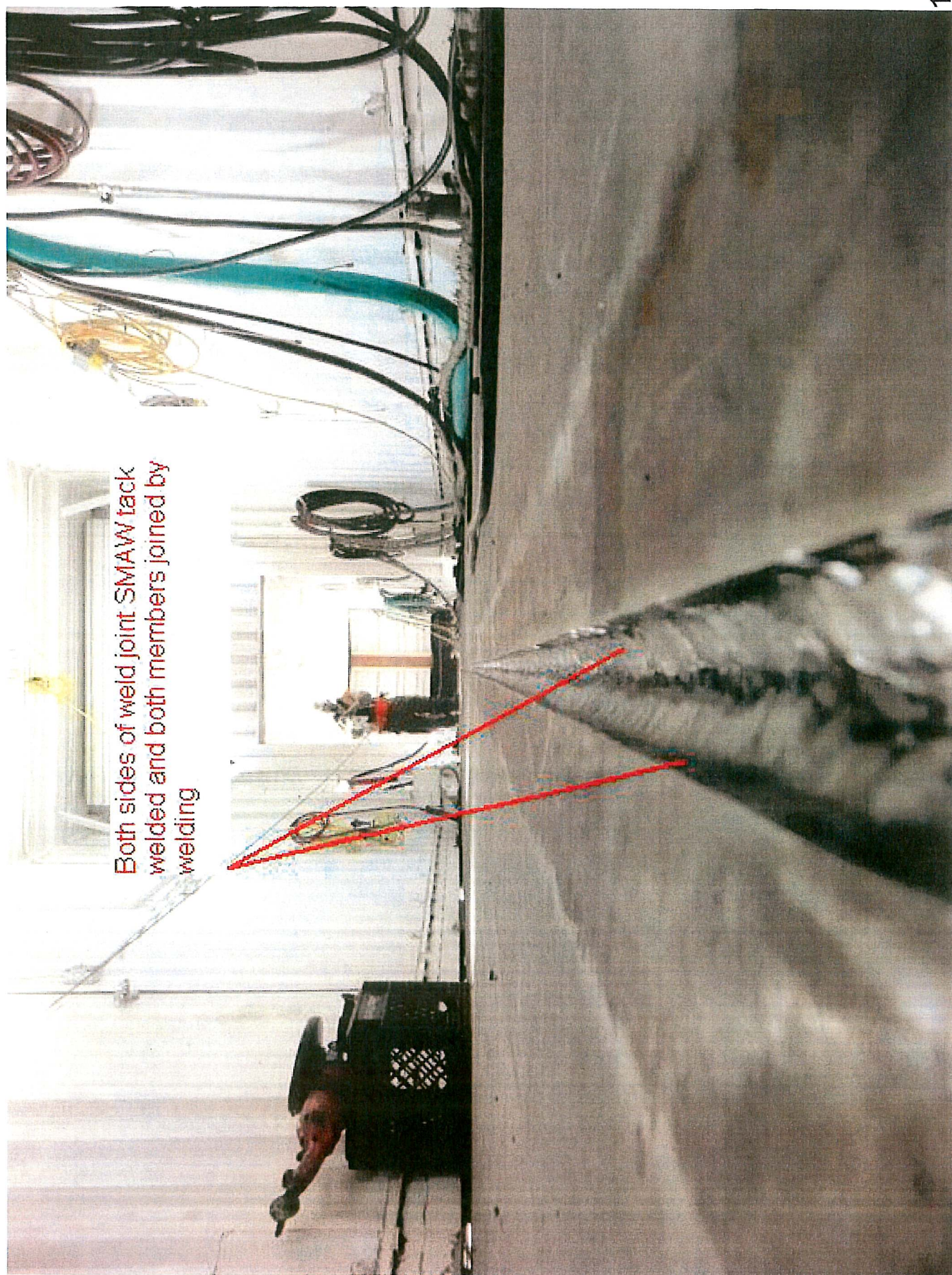


San Francisco-Oakland Bay Bridge: Self Anchored Suspension Span / Orthotropic Box Girder



One side of weld joint SMAW
tack welded

San Francisco-Oakland Bay Bridge: Self Anchored Suspension Span / Orthotropic Box Girder



Both sides of weld joint SMAW tack welded and both members joined by welding



Submerged arc welding
(SAW) root pass

San Francisco-Oakland Bay Bridge: Self Anchored Suspension Span / Orthotropic Box Girder

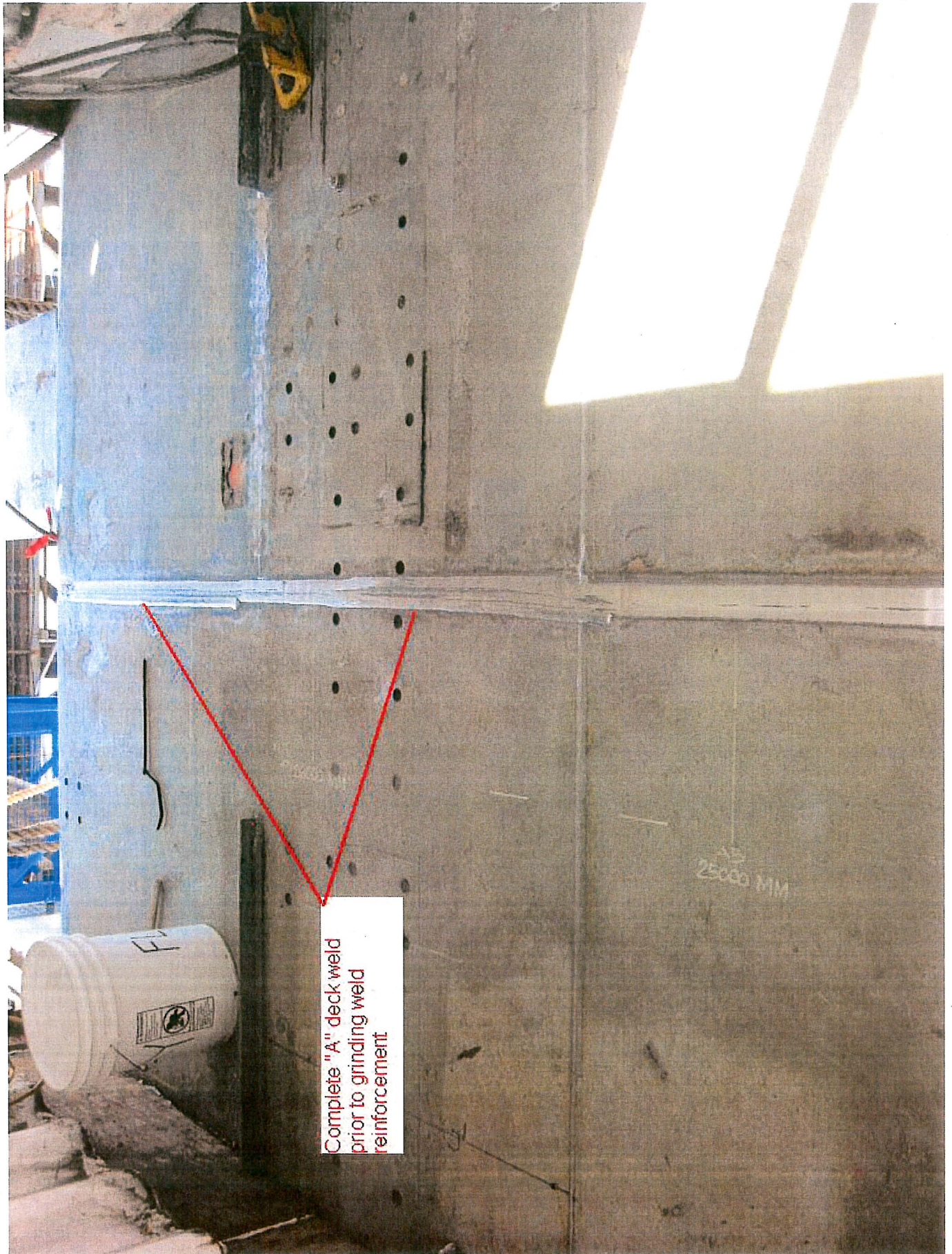


Smith Emery Quality Control
Magnetic Particle Test (MT)
SAW root pass



SAW fill pass in progress.

San Francisco-Oakland Bay Bridge: Self Anchored Suspension Span / Orthotropic Box Girder



Complete "A" deck weld
prior to grinding weld
reinforcement

25000 MM



Smith Emery QC Inspectors perform Ultrasonic testing (UT) on finished weld.

APPENDIX C

OFFSET MAPS

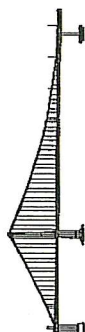


**American
Bridge**

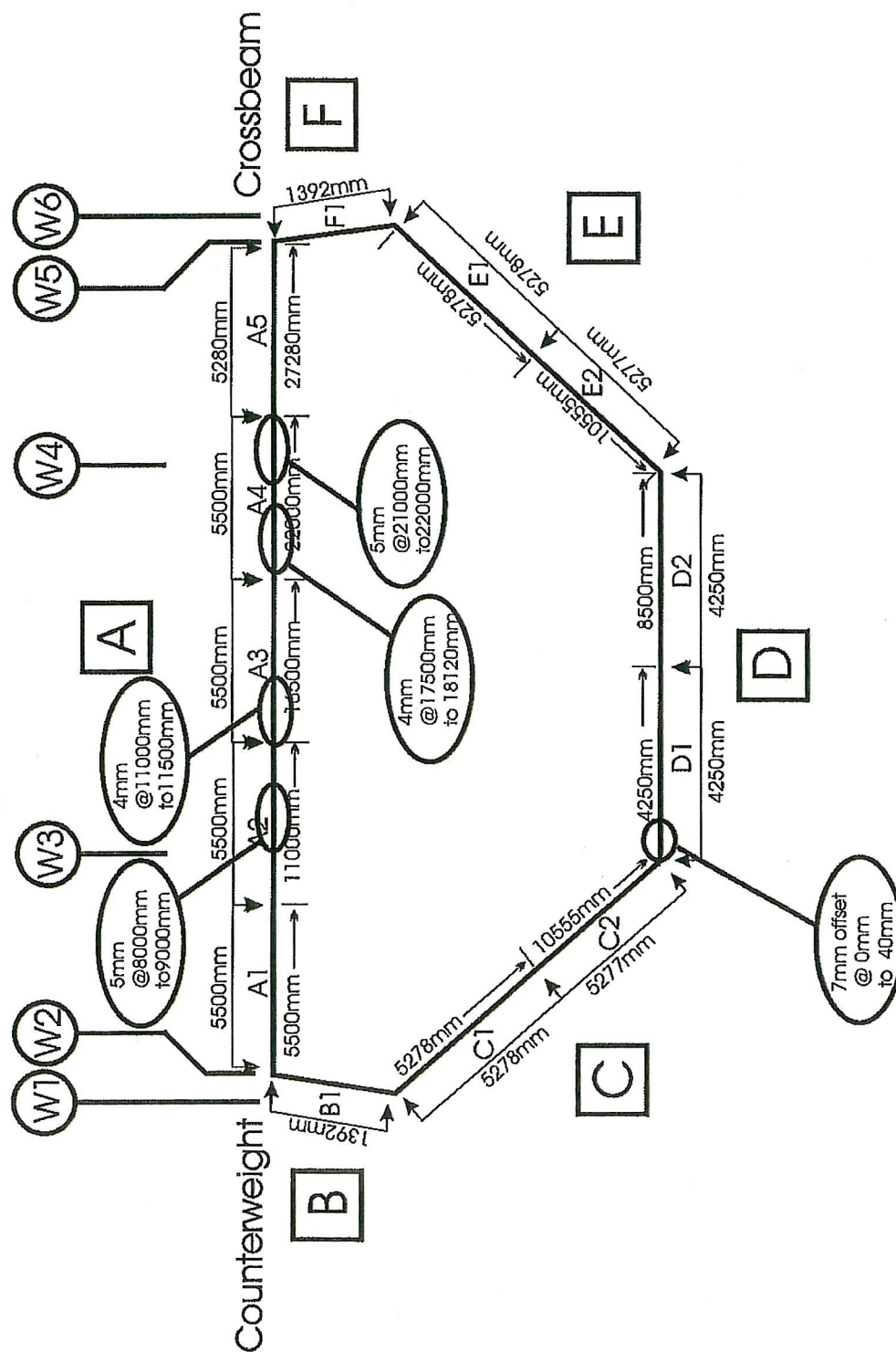
FLUOR®

A JOINT VENTURE

LSA/SECO



1W-2W Offset Map



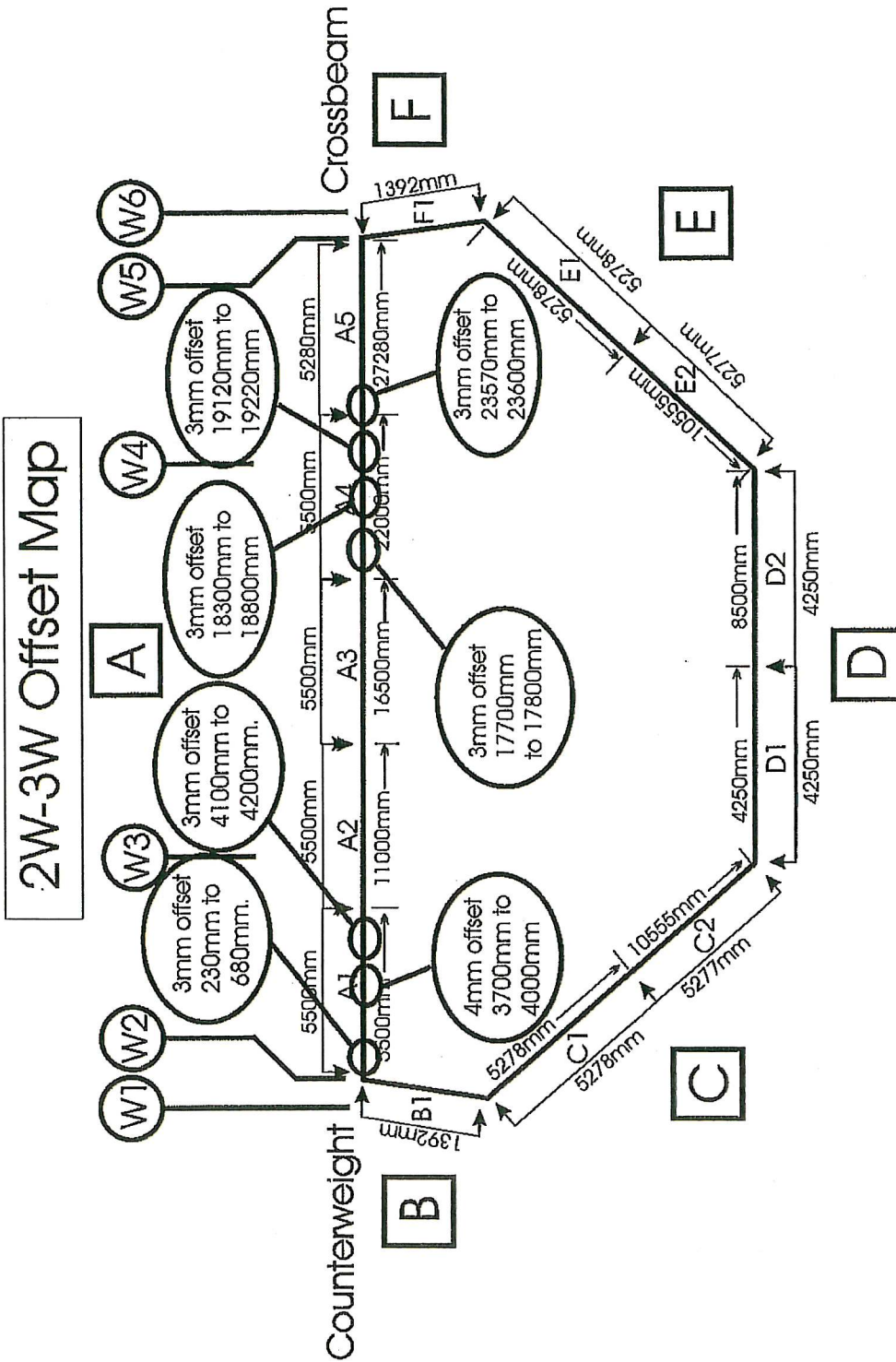


American
Bridge

FLUOR

A JOINT VENTURE

LSA/SECO



Planar Misalignment Map

COUNTERWEIGHT SIDE

WM

Planar Misalignment Map

Preliminary, Before Root Seal

CM

27280 III

X_h Axis

DEED

T

Ⓐ

③

51



Distance Measured above Offset Tolerance

८

(4) $> 6 \text{ mm} +$
 (3) $> 4 \text{ mm to } 6$
 (2) $> 2 \text{ mm to } 4$
 (1) $0 \text{ mm to } 2 \text{ mm}$

d

[illegible]

WELDON JOINT

INSPECTORS

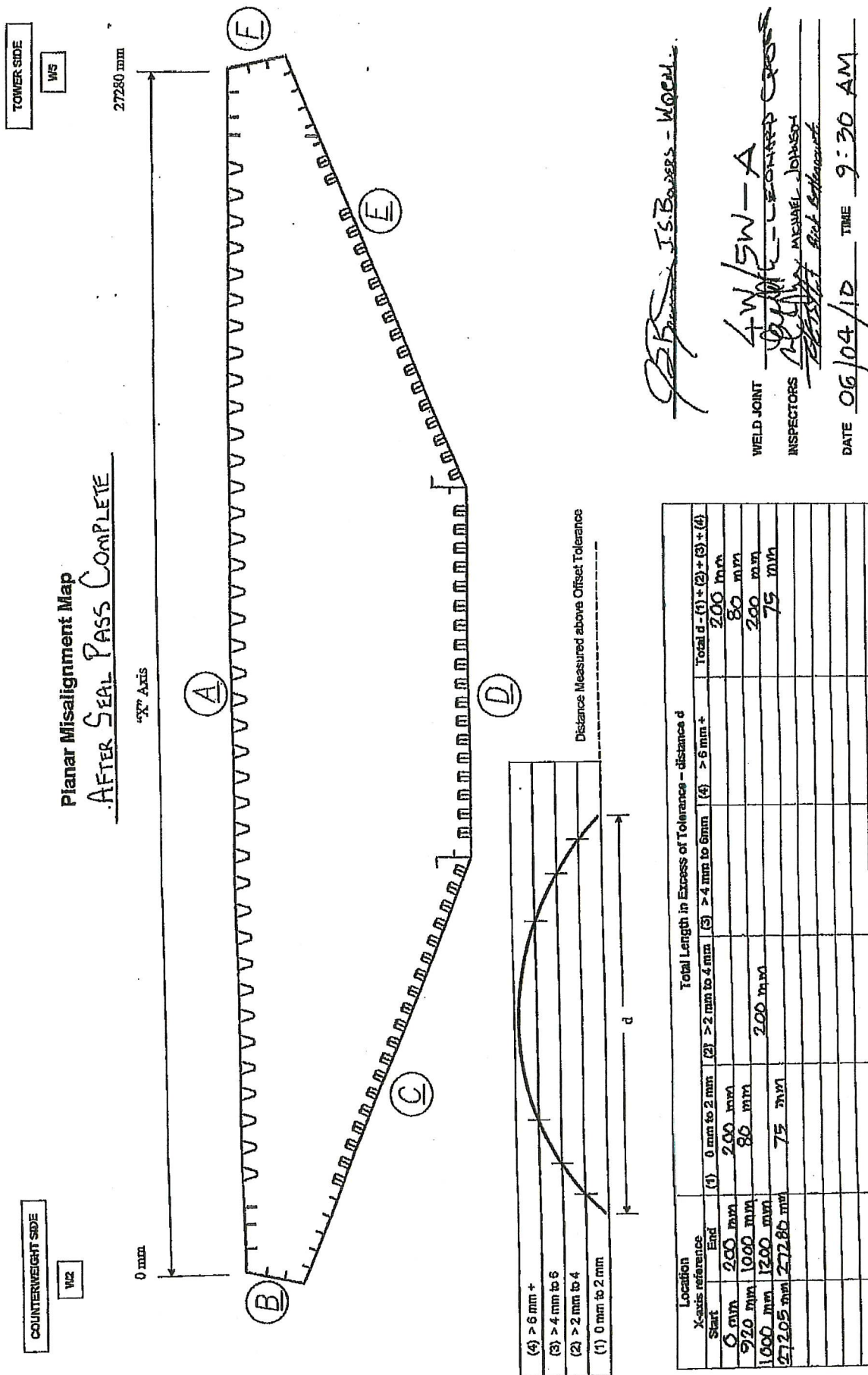
DATE _____

05/03/10

TIME

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4W/5W - A ~~SEAN'S + SEAN~~
 Julie T. ~~SEAN'S~~
 B. AQUINAS R. 807 ✓



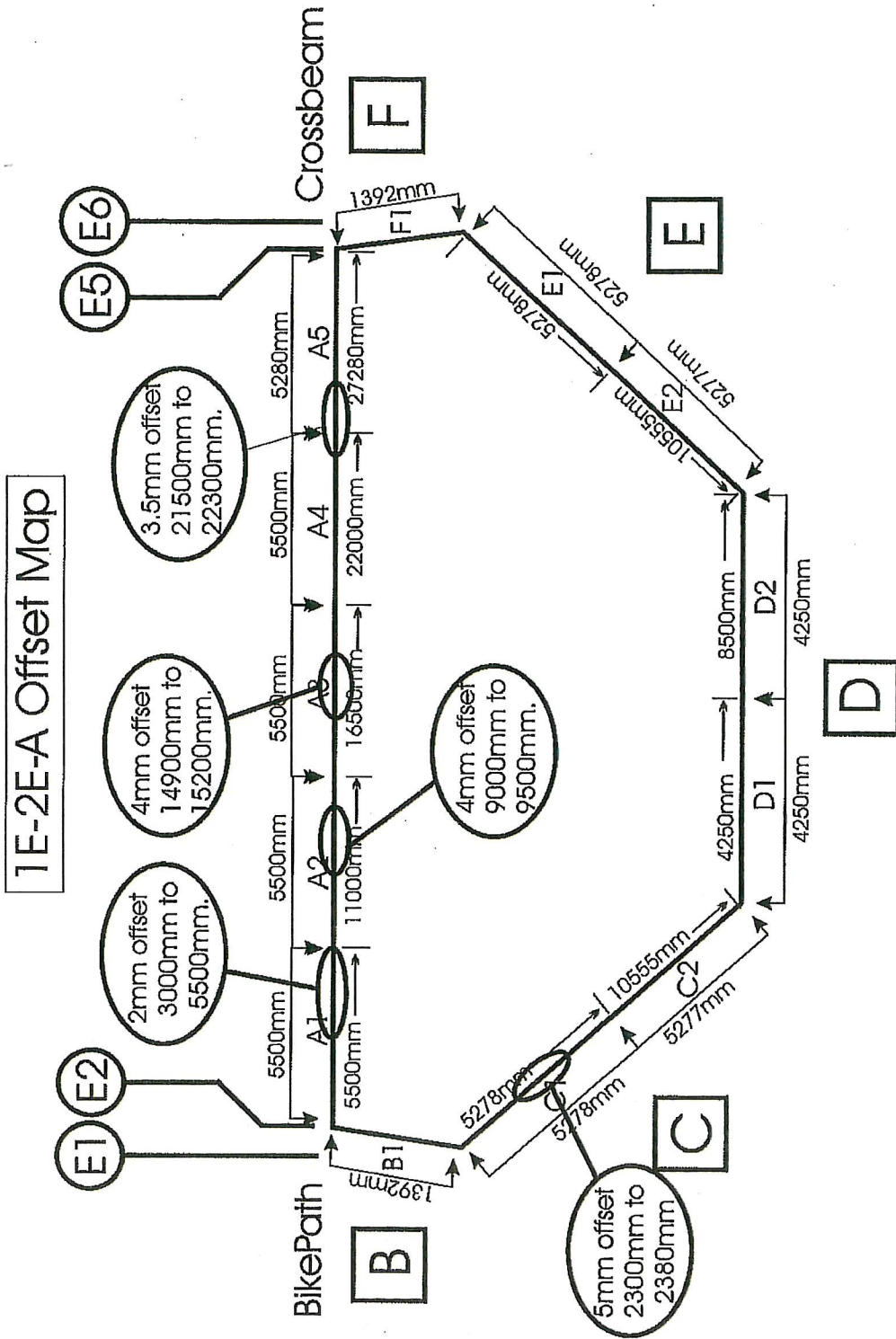


American
Bridge

FLUOR

A JOINT VENTURE

LSA/SECO





American
Bridge

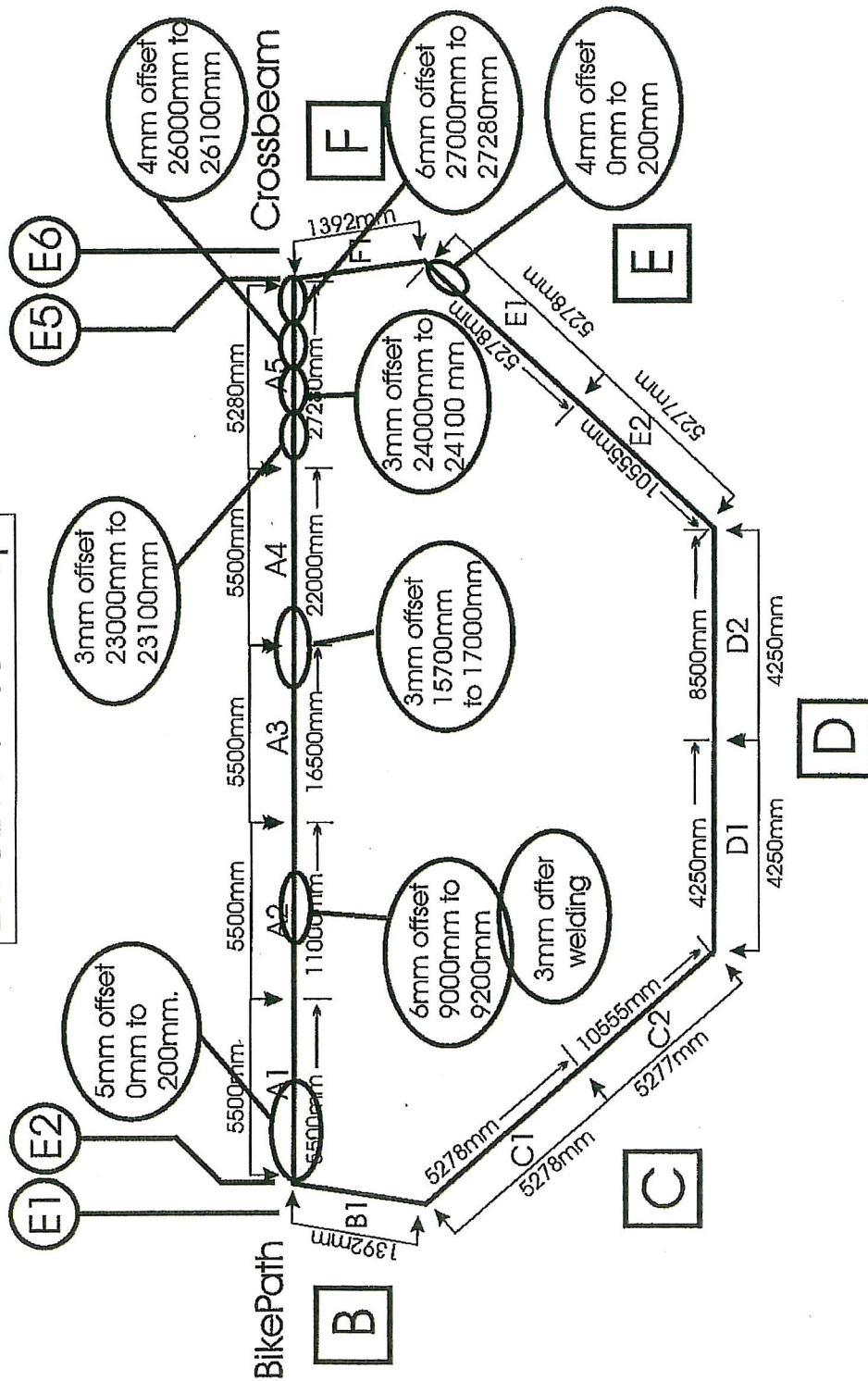
FLUOR®

A JOINT VENTURE

LSA/SECO



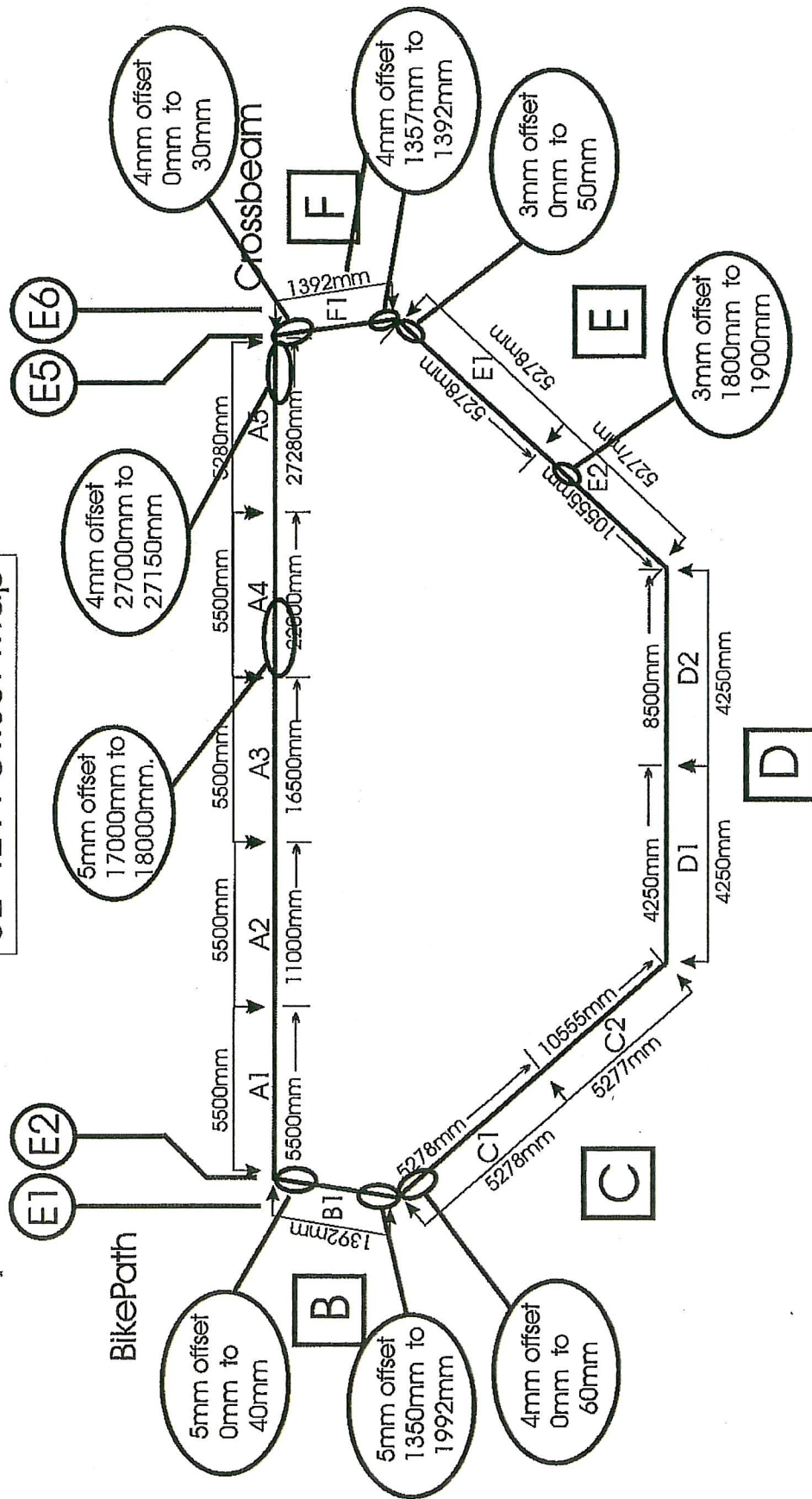
2E-3E-A Offset Map

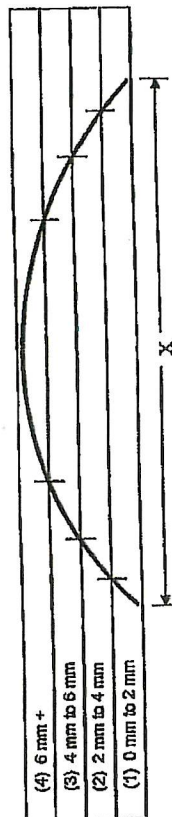
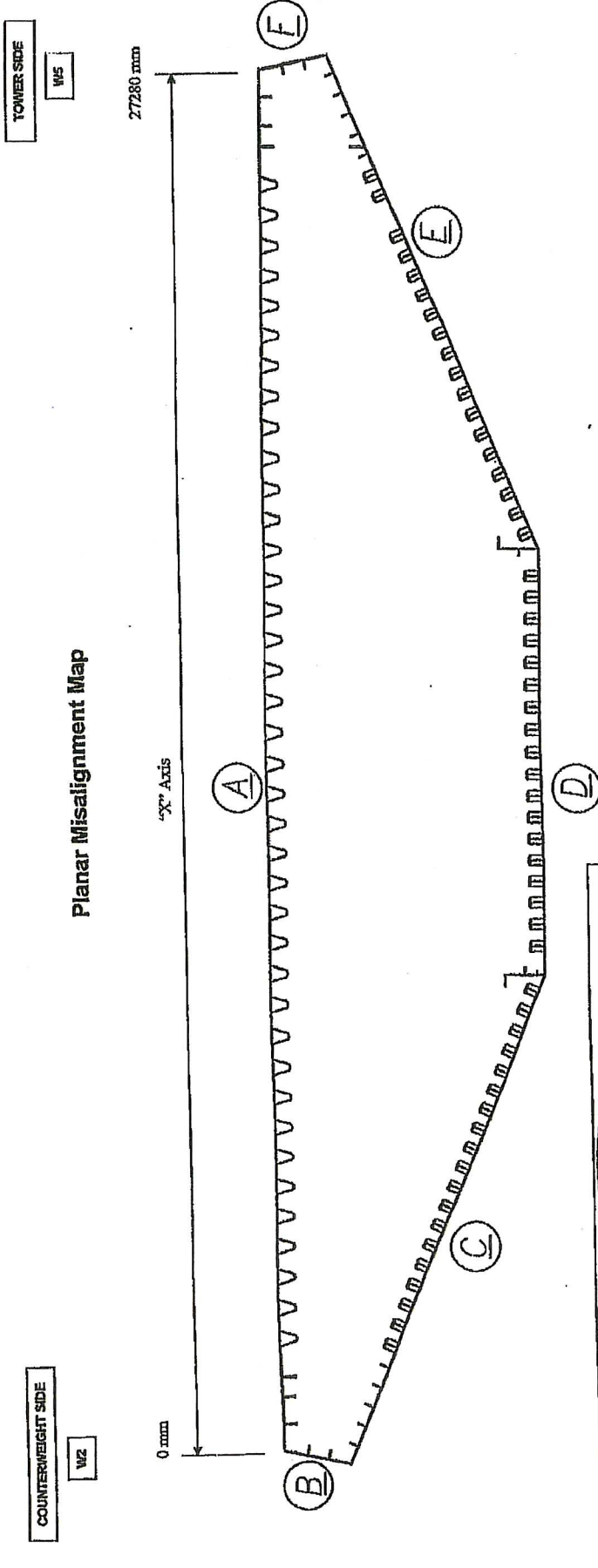




LSA/SECO

3E-4E-A Offset Map





Location		Total Length in Excess of Tolerance					
		(1) 0 mm to 2 mm	(2) 2 mm to 4 mm	(3) 4 mm to 6 mm	(4) 6 mm +	Total - (1) + (2) + (3) + (4)	
Start	End						
250	700	250 mm	0	0	0	250 mm	250 mm
700	1500	1000 mm	0	0	0	1000 mm	1000 mm
1500	3500	370 mm	0	0	0	370 mm	370 mm
3500	5100	510 mm	0	0	0	510 mm	510 mm
5100	8500	480 mm	0	0	0	480 mm	480 mm
8500	10250	830 mm	0	0	0	830 mm	830 mm
10250	10800	425 mm	0	0	0	425 mm	425 mm
10800	1175	160 mm	0	0	0	160 mm	160 mm
1175	17180	0	0	0	0	0	0
17180	17820	0	0	0	0	0	0

WELD JOINT WN-5E-6E-A

INSPECTORS C. J. Kankpiki
Rich Belmont

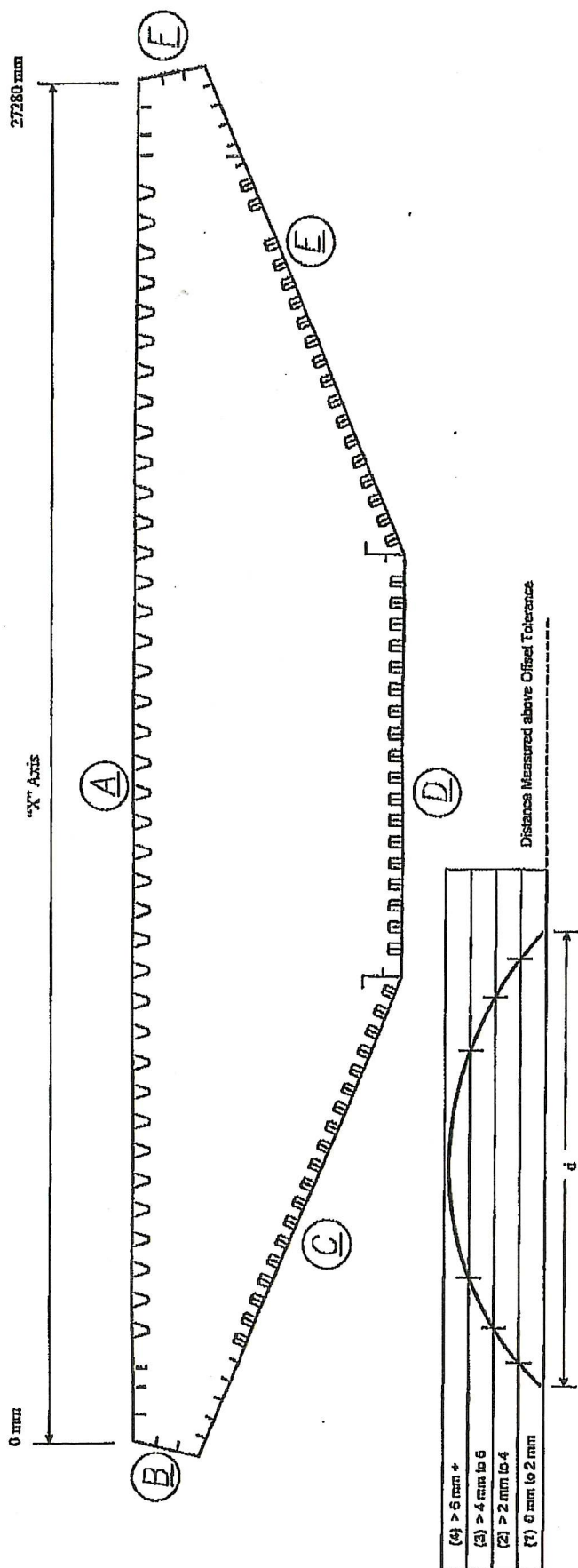
DATE 5/14/10 TIME 1:30 PM

53

Planar Misalignment Map

BIKEPATH SIDE

23



Location		Total Length in Excess of Tolerance -- distance d					
X-axis reference	End	(1)	(2)	(3)	(4)	Total d - (1) + (2) + (3) + (4)	
		0 mm to 2 mm	> 2 mm to 4 mm	> 4 mm to 6 mm	> 6 mm +		
17910	18080	170 MM	0	0	0	170 MM	
22380	22545	165 MM	0	0	0	165 MM	
22710	23000		250 MM	0	0	250 MM	
23000	23150	150 MM	0	0	0	150 MM	
23310	23480	170 MM	0	0	0	170 MM	
27045	27220	175 MM	0	0	0	175 MM	

WELDON JOINT

INSPECTORS

DATE _____

51410

TIME

ME 03:1

WZ-SE-CE-4
F. Kautzke
Bischoffshausen / Eifel

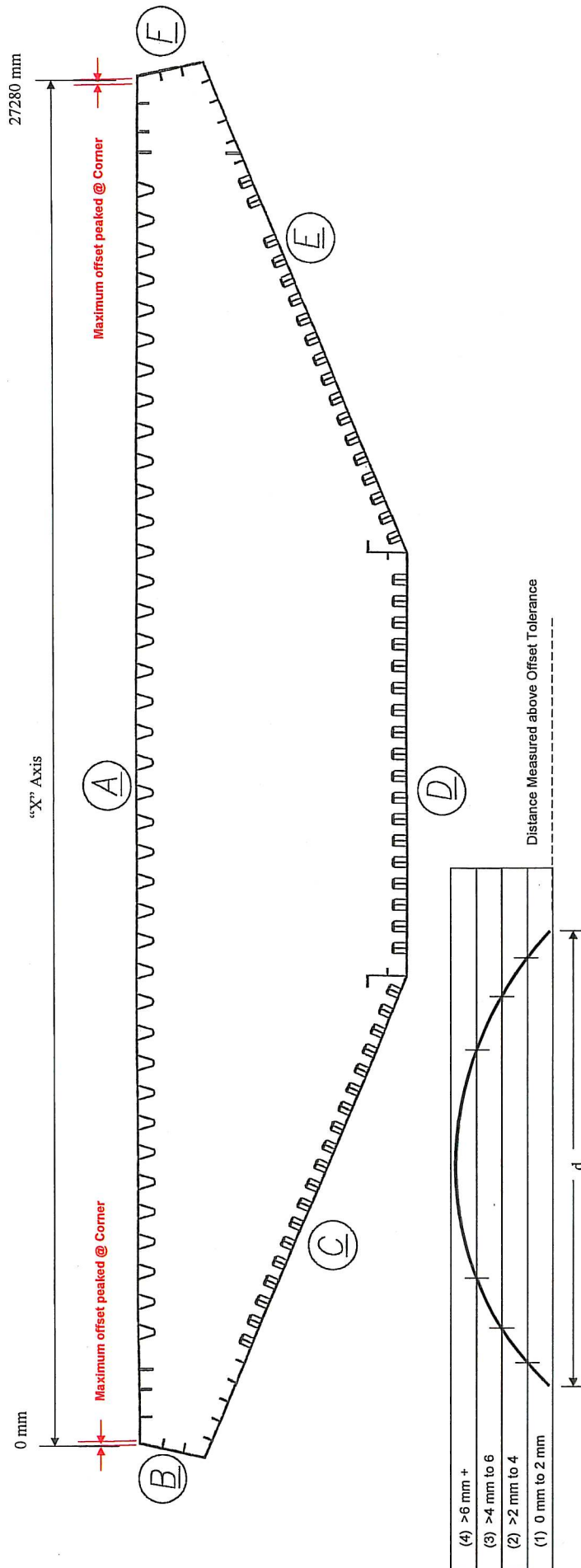
BIKEPATH SIDE

TOWER SIDE

W2

W5

Planar Misalignment Map OBG 6E to 7E



Location	Total Length in Excess of Tolerance - distance d						
	(1) 0 mm to 2 mm	(2) >2 mm to 4 mm	(3) >4 mm to 6 mm	(4) >6 mm +	Total d - (1) + (2) + (3) + (4)		
Start	0	280	140mm	140mm	0	0	280mm
End	27215	27280	15mm	50mm	0	0	65mm

WELD JOINT 6E-7E-A

INSPECTORS Leonard Cross - ABF/LSA
Rick Bettencourt - CT

DATE 07-Sept-2010 TIME 12:30

APPENDIX D

DECK PLATE SECONDARY STRESS CALCULATIONS



**SAN FRANCISCO OAKLAND BAY BRIDGE
EAST SPAN SEISMIC SAFETY PROJECT
SELF-ANCHORED SUSPENSION BRIDGE
(SUPERSTRUCTURE AND TOWER)**

Project No.: 660110
Designed By: Kevin Smith
Date: 10/29/2010
Page: 1 of 4
Revision: 3

Contract No. 04-0120F4 Bridge No. 34-0006L/R District 04 County SF Route 80 Kilometer Post 13.2 / 13.9

BOX GIRDER DECK PLATE OFFSET SECONDARY STRESS CALCULATIONS

PURPOSE:

Determine the secondary flexural stress in the OBG deck plate transverse splices due to planar misalignments in the deck plate.

DESIGN APPROACH:

Combine global and local axial stresses in the deck plate due to dead and live load. Detailed calculations are provided below for OBG field splice 1E. Resulting stresses for all of the OBG field splices are provided in the attached table.

DESIGN STANDARD:

AASHTO LRFD Bridge Design Specifications, Second Edition, 1998, Section 6.6.1.2 for Load-Induced Fatigue.
Self-Anchored Suspension Bridge Design Criteria, dated 04/08/02.
Caltrans Bridge Design Specifications, February 2004.

REFERENCES:

Submittal ABF-SUB-000457R02 and ABF-SUB-001376R00
Contract Plan Sheet Nos. 643, 738 and 741.

CALCULATIONS:

Dead Load Compression in Box Girder at Field Splice 1E:

$$\begin{array}{ll}
 P_{DL} := -200 \text{ MN} & P_{DL} = -200000 \cdot \text{kN} \quad (\text{Per Drawing 738}) \\
 A_{gross} := 2.34351 \text{ m}^2 & A_{gross} = 2.344 \cdot \text{m}^2 \quad (\text{Per Submittal 457R02}) \\
 f_{DLC} := \frac{P_{DL}}{A_{gross}} & f_{DLC} = -85.34 \cdot \text{MPa}
 \end{array}$$

Dead Load Moment in Box Girder at Field Splice 1E:

$$\begin{array}{ll}
 M_{DL} := -55.801 \text{ MN} \cdot \text{m} & M_{DL} = -55.8 \cdot \text{MN} \cdot \text{m} \quad (\text{Per Submittal 1376R00}) \\
 c_{y.1E} := 2.365 \text{ m} & c_{y.1E} = 2.365 \cdot \text{m} \quad (\text{Per Submittal 457R02}) \\
 I_{x.gross.1E} := 11.785 \cdot \text{m}^4 & I_{x.gross.1E} = 11.785 \cdot \text{m}^4 \quad (\text{Per Submittal 457R02}) \\
 f_{DLM} := \frac{M_{DL} \cdot c_{y.1E}}{I_{x.gross.1E}} & f_{DLM} = -11.2 \cdot \text{MPa}
 \end{array}$$

Live Load Compression in Box Girder at Field Splice 1E:

$$P_{LL} := -20MN$$

$$P_{DL} = -200000 \cdot kN \quad (\text{Per WDC Discussions})$$

$$f_{LLC} := \frac{P_{LL}}{A_{gross}}$$

$$f_{LLC} = -8.53 \cdot MPa$$

Minimum Live Load Moment in Box Girder at Field Splice 1E:

$$M_{LL.min} := -97.142MN \cdot m$$

$$M_{LL.min} = -97.14 \cdot MN \cdot m \quad (\text{Per WDC Discussions})$$

$$f_{LLM.min} := \frac{M_{LL.min} \cdot c_{y.1E}}{I_{x.gross.1E}}$$

$$f_{LLM.min} = -19.49 \cdot MPa$$

Maximum Live Load Moment in Box Girder at Field Splice 1E:

$$M_{LL.max} := 85MN \cdot m$$

$$M_{LL.max} = 85 \cdot MN \cdot m \quad (\text{Per WDC Discussions})$$

$$f_{LLM.max} := \frac{M_{LL.max} \cdot c_{y.1E}}{I_{x.gross.1E}}$$

$$f_{LLM.max} = 17.06 \cdot MPa$$

Minimum Local Live Load Moment in Deck Plate:

$$P_{wheel} := -71kN$$

$$P_{wheel} = -71 \cdot kN \quad (\text{Per SAS Design Criteria Dated 04/08/02})$$

$$IM := 30\%$$

$$(\text{Per SAS Design Criteria Dated 04/08/02})$$

$$a := 312.5mm$$

$$e := 287.5mm$$

$$width_{eff} := 1.3(a + e)$$

$$width_{eff} = 0.78 \cdot m \quad (\text{Per AASHTO LRFD, 1998})$$

$$M_{LLL.min} := -28.447kN \cdot m$$

$$(\text{Per Service Load Influence Lines})$$

$$c_{rib} := 83.1mm$$

$$c_{rib} = 0.083 \cdot m$$

$$I_{rib} := .00032534 \cdot m^4$$

$$I_{rib} = 0.0003253 \cdot m^4$$

$$f_{LLL.min} := \frac{(1 + IM) \cdot M_{LLL.min} \cdot c_{rib}}{I_{rib}}$$

$$f_{LLL.min} = -9.45 \cdot MPa$$

$$f_{LLL.min.vonMises} := -41MPa$$

Total local live load stress calculated from FEA analysis. Includes 30% impact factor.

Maximum Local Live Load Moment in Deck Plate:

$$M_{LLL.max} := 11.527 \text{ kN}\cdot\text{m}$$

(Per Service Load Influence Lines)

$$f_{LLL.max} := \frac{[(1 + IM) \cdot M_{LLL.max} \cdot c_{rib}]}{I_{rib}} \quad f_{LLL.max} = 3.83 \cdot \text{MPa}$$

$$f_{LLL.max.vonMises} := 29 \text{ MPa}$$

Total local live load stress calculated from FEA analysis. Includes 30% impact factor.

Total Service Load Deck Plate Axial Stress:

$$f_{sum.max} := f_{DLC} + f_{DLM} + f_{LLC} + f_{LLM.max} + f_{LLL.max.vonMises} = -59.02 \cdot \text{MPa}$$

$$f_{sum.min} := f_{DLC} + f_{DLM} + f_{LLC} + f_{LLM.min} + f_{LLL.min.vonMises} = -165.57 \cdot \text{MPa}$$

Evaluate a 100mm Unit Width of Deck Plate at Splice 1E:

$$w_{100} := 100 \text{ mm}$$

$$t_{DP} := 20 \text{ mm}$$

$$f_{sec.min} := f_{DLC} + f_{DLM} + f_{LLC} + f_{LLM.min} + f_{LLL.min} = -134.01 \cdot \text{MPa}$$

$$P_{100.min} := f_{sec.min} \cdot w_{100} t_{DP}$$

$$P_{100.min} = -268.03 \cdot \text{kN}$$

Calculate Secondary Stresses Due to the Axial Demand and the Plate Misalignment at Splice 1E:

$$ecc := 7 \text{ mm}$$

$$M_{ecc.min} := \frac{P_{100.min} \cdot ecc}{2}$$

$$M_{ecc.min} = -0.938 \cdot \text{kN}\cdot\text{m}$$

$$Z_{DP} := \frac{w_{100} t_{DP}^2}{4}$$

$$Z_{DP} = 10 \cdot \text{cm}^3$$

$$f_{secondary} := \frac{M_{ecc.min}}{Z_{DP}}$$

$$f_{secondary} = -93.81 \cdot \text{MPa}$$

Plastic Section Modulus
Used for Weak Axis
Secondary Flexural Stresses

$$f_{total.min} := f_{sum.min} + f_{secondary}$$

$$f_{total.min} = -259.38 \cdot \text{MPa}$$

Calculate Tensile Fatigue Stress from Fatigue Truck Load:

$$LF := 0.75$$

$$IM := 15$$

$$M_{fatigue} := LF \cdot (27.114 \text{ kN} \cdot \text{m}) \cdot \left(1 + \frac{IM}{100}\right)$$

$$M_{fatigue} = 23.39 \cdot \text{kN} \cdot \text{m} \quad (\text{Per Fatigue Load Influence Lines})$$

$$c_{rib} = 0.083 \cdot \text{m}$$

$$I_{rib} = 0.0003253 \cdot \text{m}^4$$

$$f_{fatigue} := \frac{M_{fatigue} \cdot c_{rib}}{I_{rib}}$$

$$f_{fatigue} = 5.97 \cdot \text{MPa}$$

Fatigue Stress in Tension

$$f_{fatigue, vonMises} := 19.24 \text{ MPa}$$

Total local live load fatigue stress calculated from FEA analysis. Includes 15% impact factor and 0.75 load factor.

Calculate Secondary Stresses Due to the Fatigue Truck Load:

$$w_{100} := 100 \text{ mm}$$

$$t_{DP, fatigue} := 20 \text{ mm}$$

$$P_{100, fatigue} := f_{fatigue} \cdot w_{100} \cdot t_{DP, fatigue}$$

$$P_{100, fatigue} = 11.95 \cdot \text{kN}$$

$$ecc := 7 \text{ mm}$$

$$M_{ecc, fatigue} := \frac{P_{100, fatigue} \cdot ecc}{2}$$

$$M_{ecc, fatigue} = 0.042 \cdot \text{kN} \cdot \text{m}$$

$$f_{secondary, fatigue} := \frac{M_{ecc, fatigue}}{Z_{DP}}$$

$$f_{secondary, fatigue} = 4.18 \cdot \text{MPa}$$

$$f_{total, fatigue} := f_{fatigue, vonMises} + f_{secondary, fatigue}$$

$$f_{total, fatigue} = 23.42 \cdot \text{MPa}$$

$$\Delta F_{th} := 110 \text{ MPa}$$

For Detail Category B

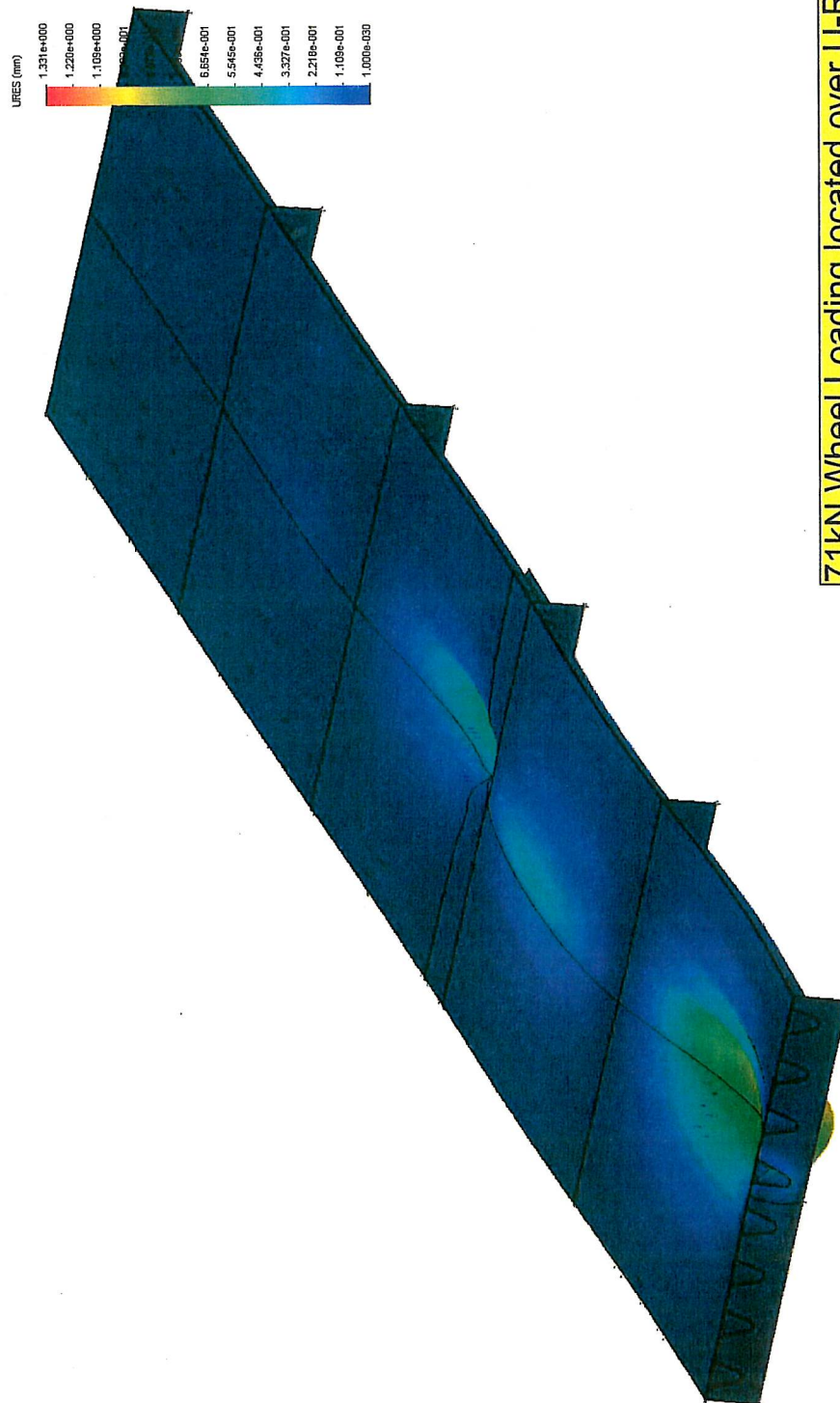
$$\Delta F_n := \frac{\Delta F_{th}}{2}$$

$$\Delta F_n = 55 \cdot \text{MPa}$$

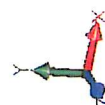
$$DC := \text{if} \left(f_{total, fatigue} < \frac{|f_{DLC} + f_{DLM}|}{2}, \text{"Fatigue Need Not Be Considered"}, \frac{|f_{total, fatigue}|}{\Delta F_n} \right)$$

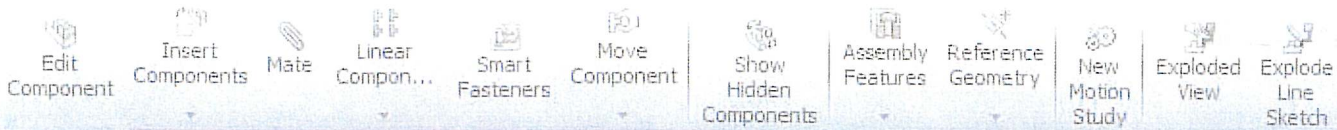
$$DC = \text{"Fatigue Need Not Be Considered"}$$

Model name: Deck Assembly Fine Mesh
 Study name: Fatigue-Load 3
 Study type: Static displacement Displacement1
 Normalization scale: 1000



71kN Wheel Loading located over U-Rib.
 Wheel Area = 510mmx250mm.





Assembly Layout Sketch Evaluate Office Products



Probe Result ?



Options

- ☐ At location
☐ From sensors
☒ On selected entities

Results



Face<1>@Deck Plate 43
Face<2>@Deck Plate 43

☐ Flip edge plot

Update

Location	Value (N/mm ² (MPa))
1063	15.8
1064	3.1
1065	11.0
1066	9.9
1067	7.4
1068	4.3
1069	2.6
1070	2.3

Summary

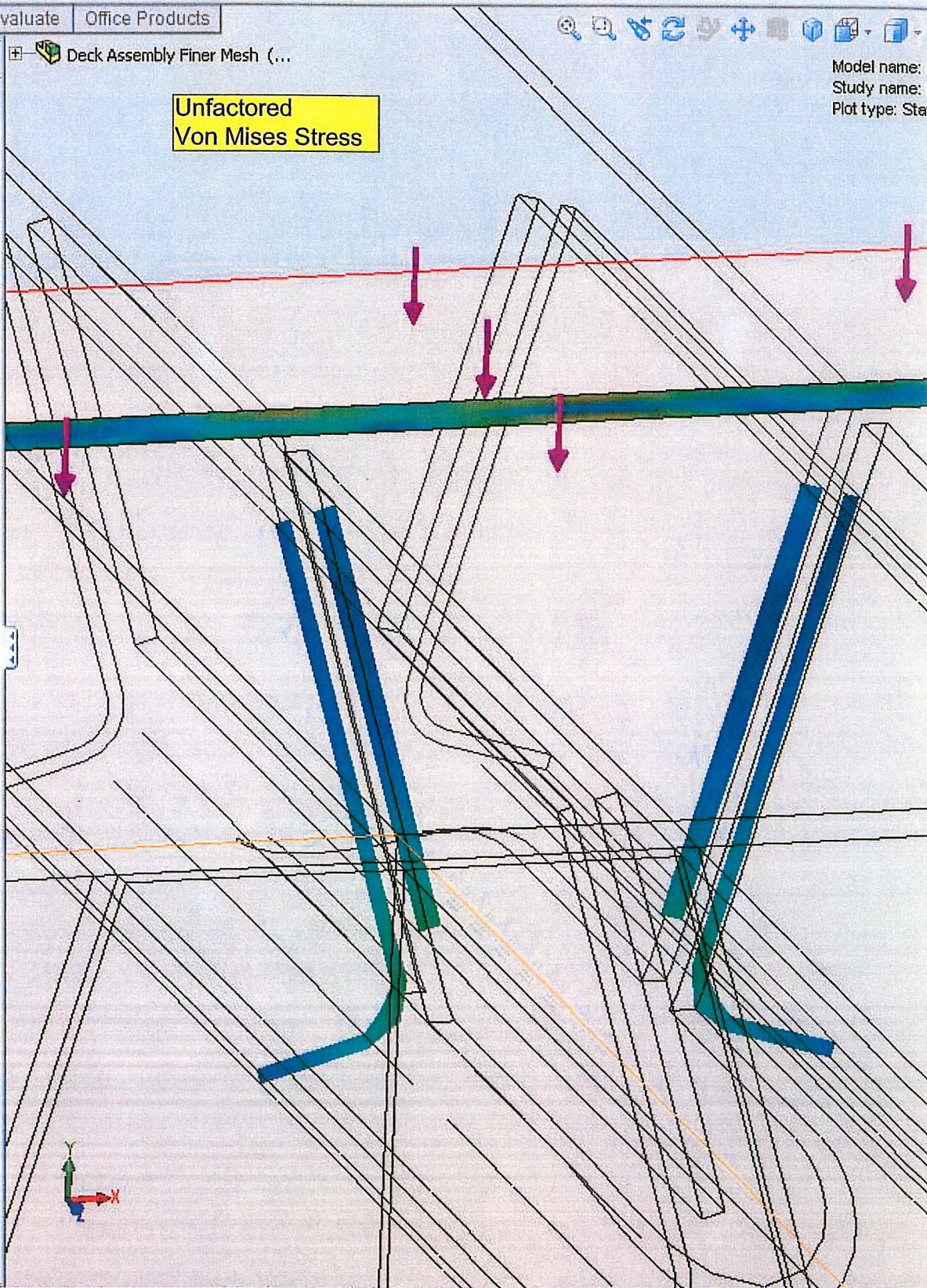
	Value	
Sum	1768.2	N/mm ² (
Avg	3.4467	N/mm ² (
Max	48.65	N/mm ² (
Min	0.0056945	N/mm ² (
RMS	7.8815	N/mm ² (

Report Options



GrabXP

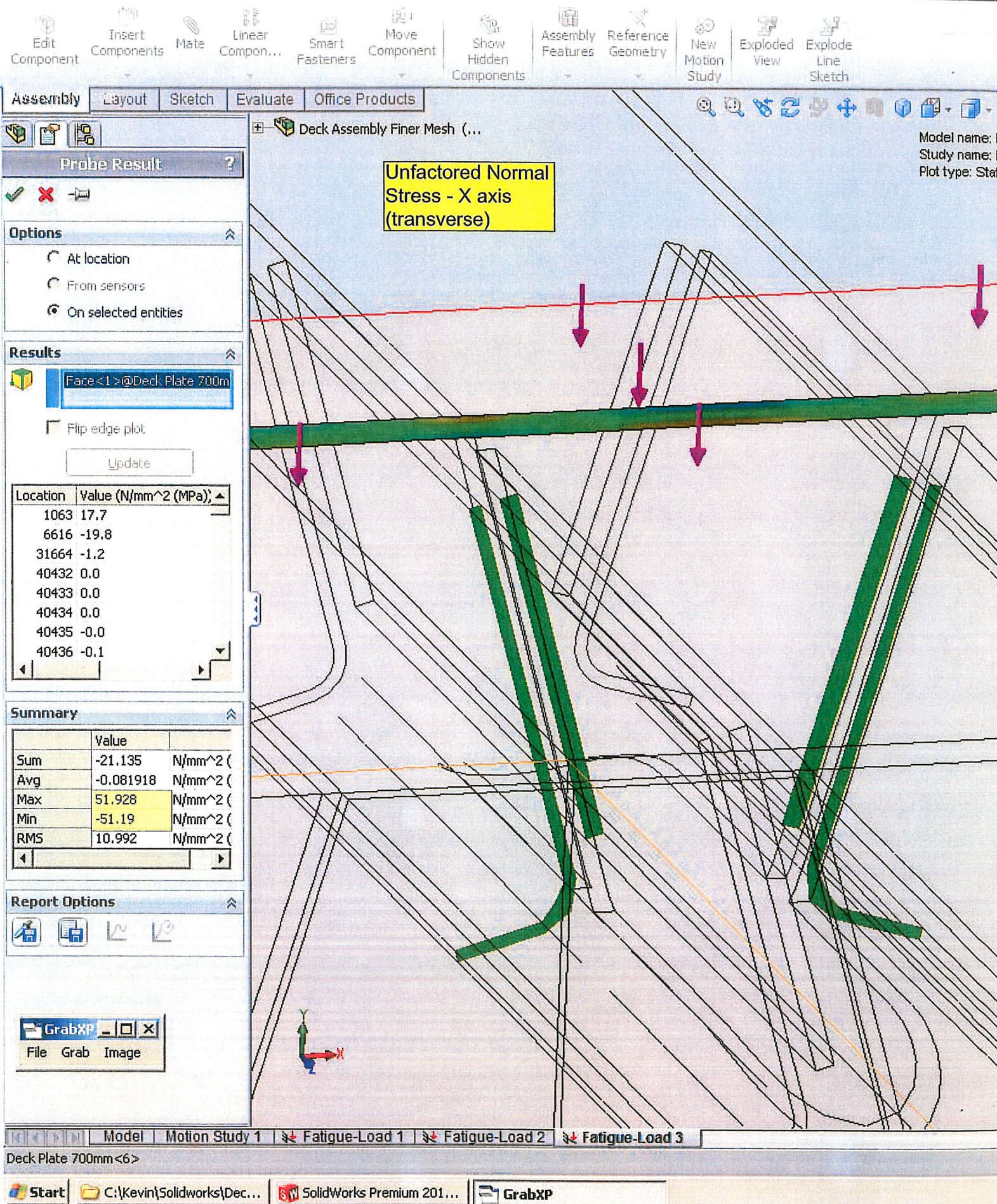
File Grab Image

**Unfactored
Von Mises Stress**

Model name: I
Study name:
Plot type: Stel

Model Motion Study 1 Fatigue-Load 1 Fatigue-Load 2 Fatigue-Load 3

Deck Plate 4300mm<6>



Edit Component Insert Components Mate Linear Component Smart Fasteners Move Component Show Hidden Components Assembly Features Reference Geometry New Motion Study Exploded View Explode Line Sketch

Assembly Layout Sketch Evaluate Office Products



Probe Result ?



Options

- ☐ At location
☐ From sensors
☒ On selected entities

Results



Face<1>@Deck Plate 700mm

☐ Flip edge plot

Update

Location	Value (N/mm ² (MPa))
1063	-0.2
6616	-0.1
31664	0.1
40432	0.0
40433	0.0
40434	-0.0
40435	0.0
40436	-0.0

Summary

	Value	
Sum	-22.518	N/mm ² (
Avg	-0.087279	N/mm ² (
Max	15.726	N/mm ² (
Min	-17.314	N/mm ² (
RMS	2.6201	N/mm ² (

Report Options

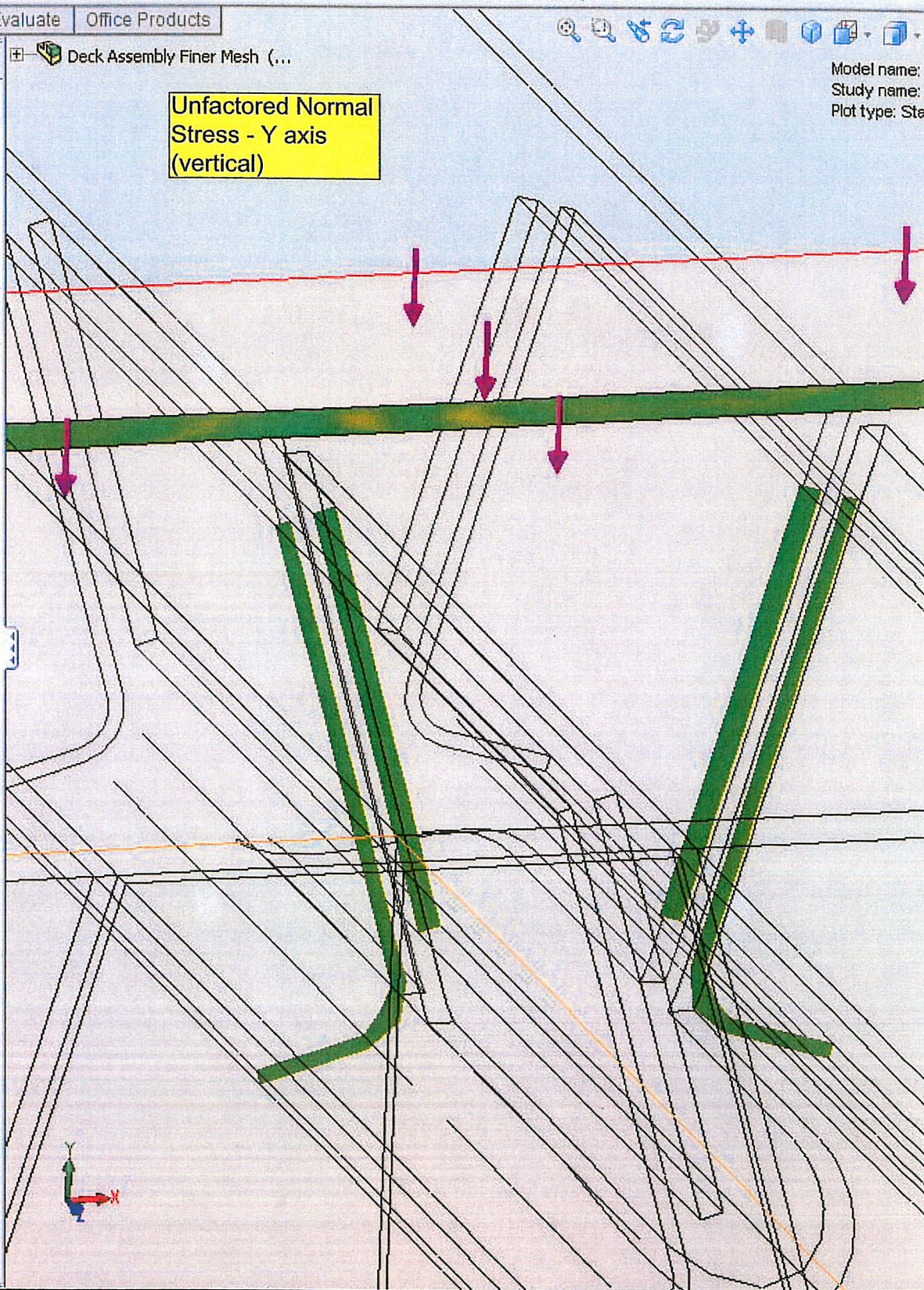


GrabXP

File Grab Image

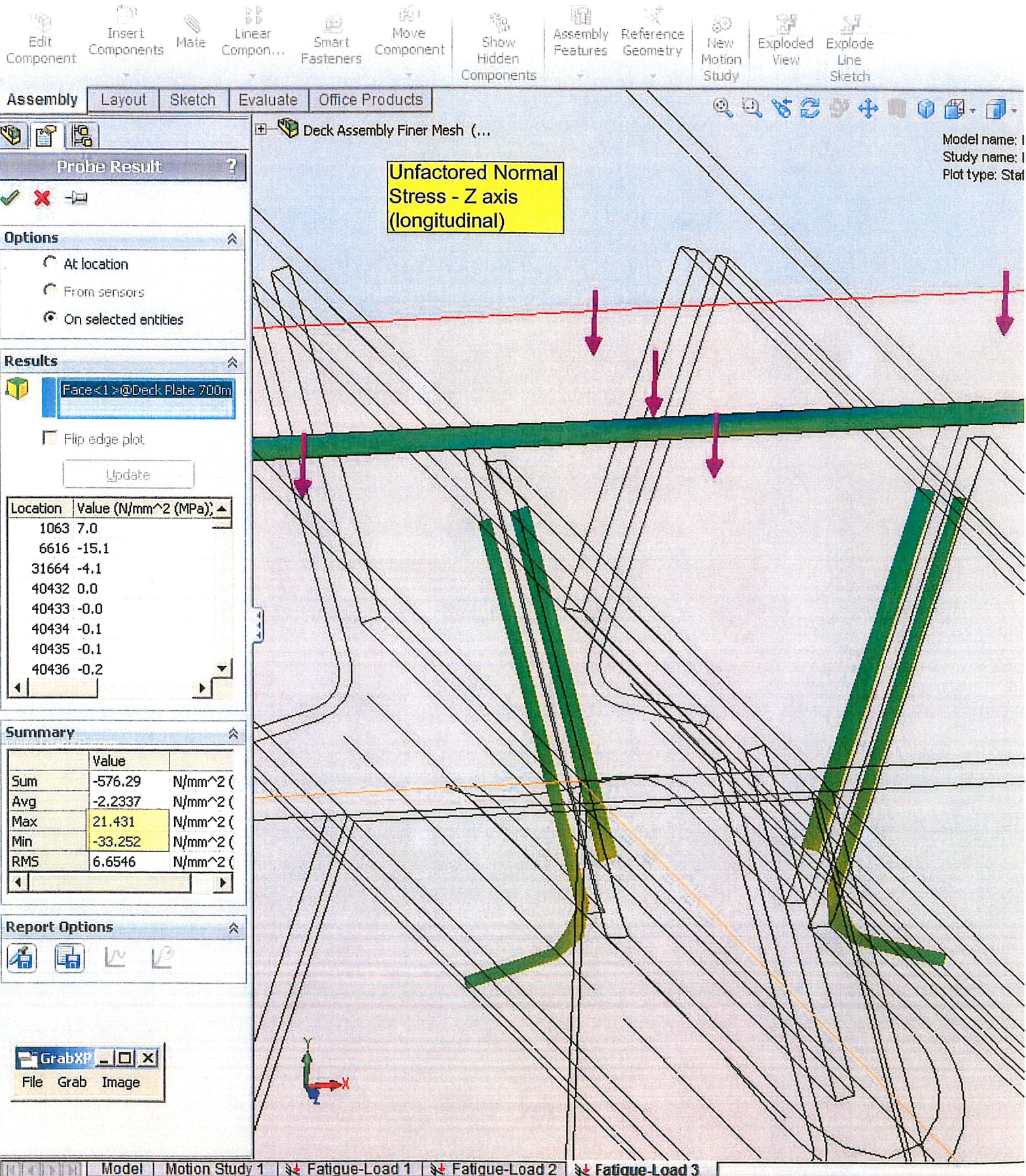
Unfactored Normal
Stress - Y axis
(vertical)

Model name: I
Study name: I
Plot type: Stal



Model Motion Study 1 Fatigue-Load 1 Fatigue-Load 2 Fatigue-Load 3

Deck Plate 700mm<6>

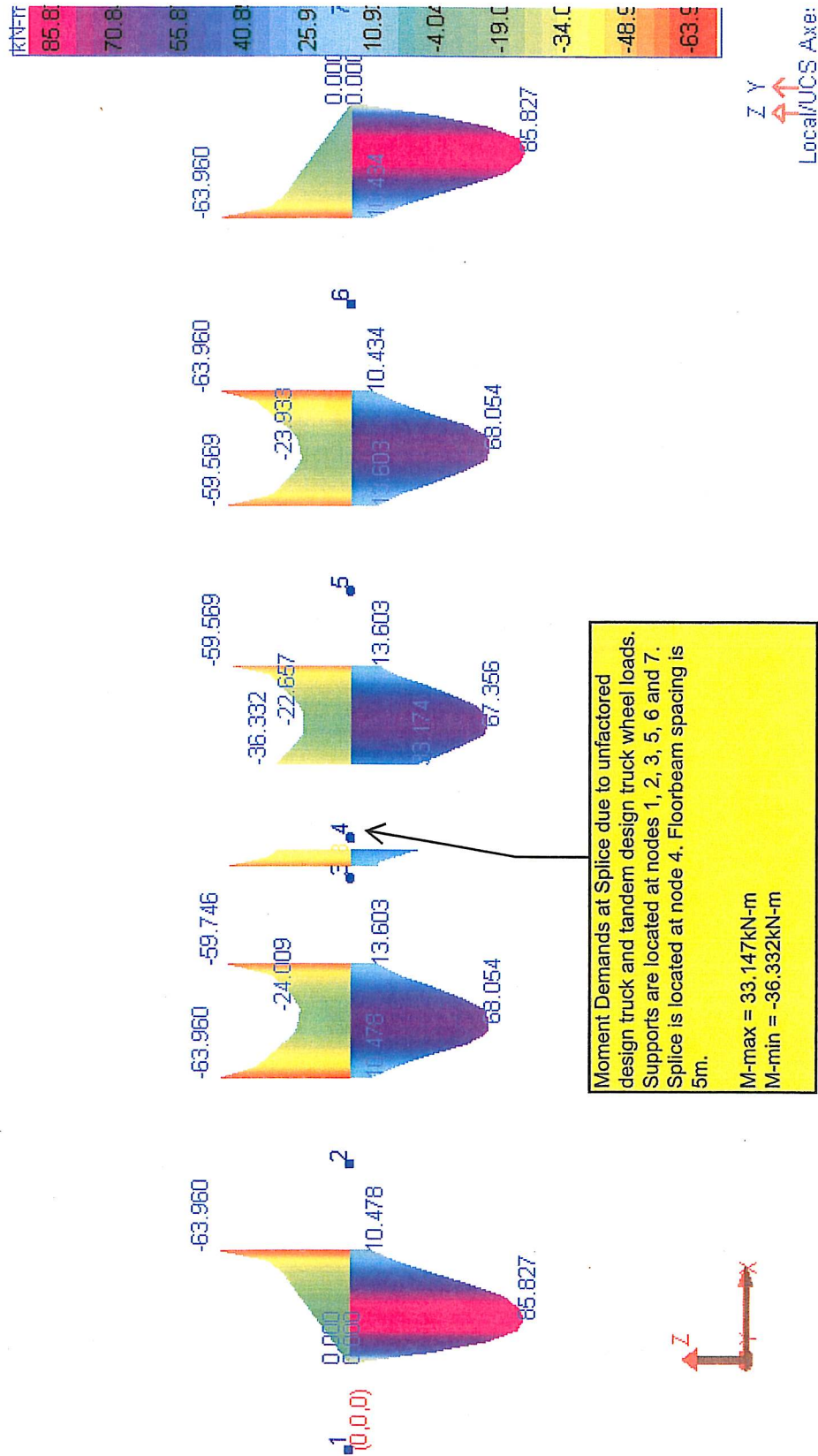


Deck Plate 700mm<6>

Graphics View 1

Zoom 1.953X

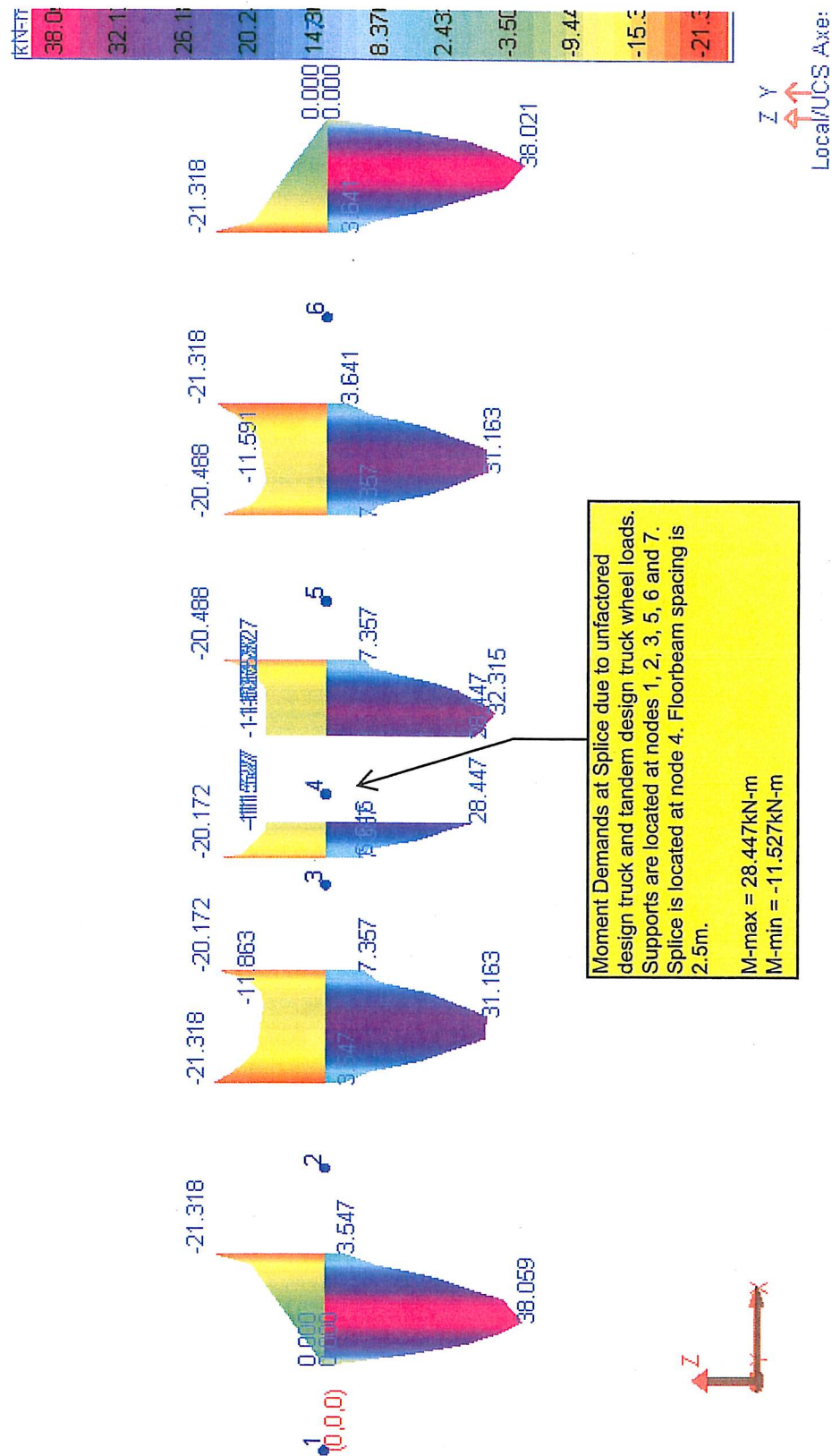
Member Forces - Moment Mz - Lane/HL-93 Design Tandem Wheel (EE), Lane/HL-93/HS20-44 Design Truck Wheel (EE)



Graphics View 1

Zoom 3.815X

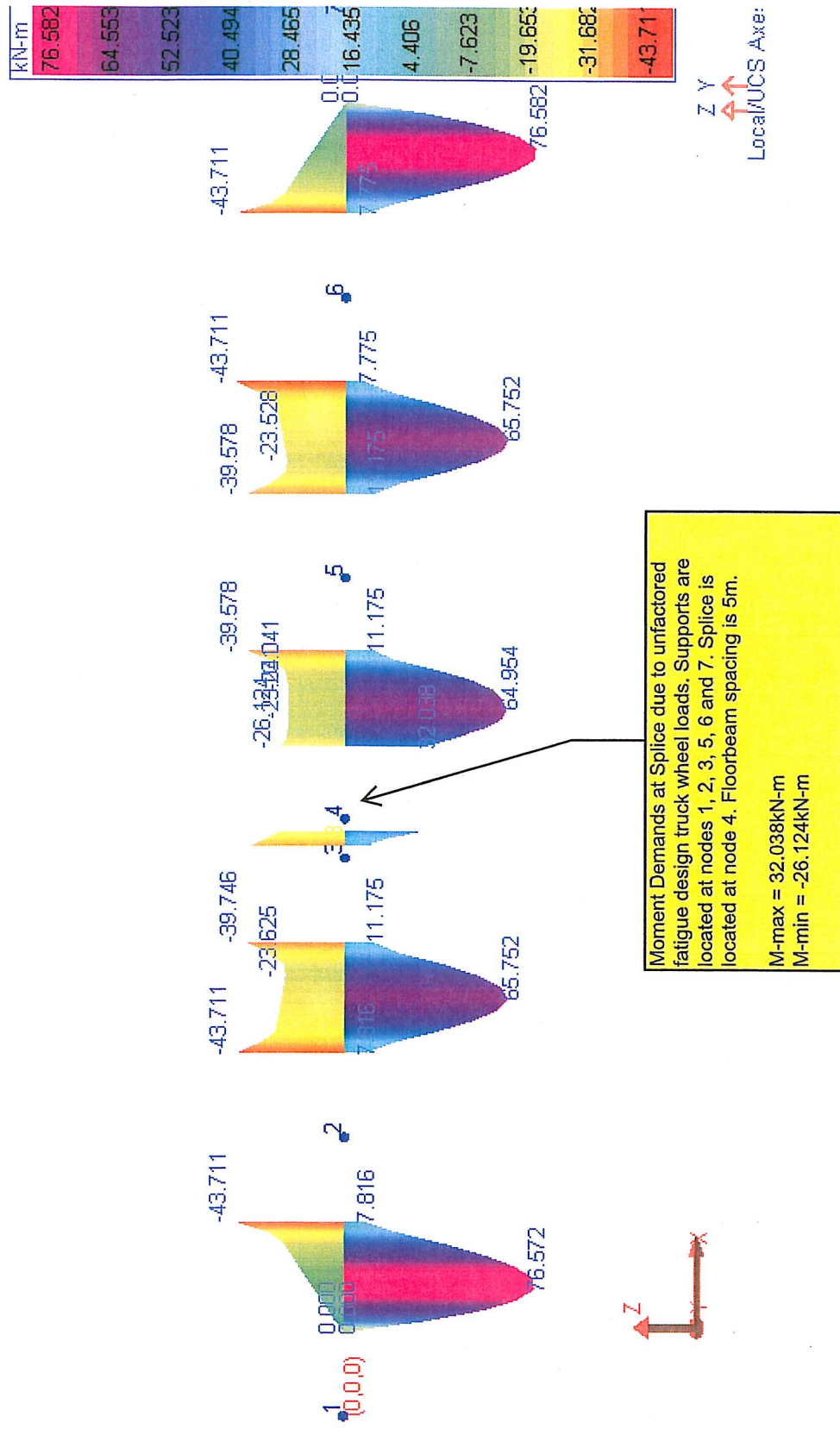
Member Forces - Moment Mz - Lane/HL-93 Design Tandem Wheel (EE), Lane/HL-93/HS20-44 Design Truck Wheel (EE)



Graphics View 1

Zoom 1.953X

Member Forces - Moment Mz - Lane/AASHTO HS-20 Fatigue Truck (EE)



Graphics View 1

Zoom 1.953X

Member Forces - Moment Mz - Lane/AASHTO Fatigue Truck (EE)

